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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

REPORT 731

Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

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Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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National Cooperative Highway Research Program (NCHRP) Project 03-95 “Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections” was led by Vanasse Hangen Brustlin, Inc. (VHB) as the prime contractor. Subcontractor assistance was provided by Wayne State University and Sam Schwartz Engineering. Dr. Hugh McGee of VHB served as the principal investigator and was supported by VHB staff members Kevin Moriarty, Kim Eccles, and Mindy Liu. Dr. Timothy Gates led the efforts of Wayne State University. Mr. Richard Retting of Sam Schwartz Engineering provided consulting services.

FOREWORD

By David A. Reynaud
Staff Officer
Transportation Research Board

NCHRP Report 731: Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections provides comprehensive and uniform guidelines for safe and efficient yellow change and all-red clearance intervals at signalized intersections. These proposed guidelines provide a framework that can be easily applied by state and local transportation agencies. This report will be of interest to safety and traffic engineers.

Red-light running is one of the most common causes of intersection crashes. Yellow and all-red interval duration is a significant factor affecting the frequency of red-light running, yet there remains no national consensus on how the yellow and all-red intervals should be timed for safe and efficient operations. The generally accepted definition of the yellow change interval is that it is a warning to motorists that the related green movement is being terminated or that a red signal indication will be exhibited immediately thereafter. Some jurisdictions supplement the yellow interval with an all-red interval to provide additional clearance time.

Studies of driver reaction times and vehicle deceleration rates used in determining appropriate yellow and all-red change intervals were conducted more than 25 years ago. Additional research was needed to consider other factors that may be important in designing change intervals including speeds, grades, vehicle types, vehicle mix, road surface conditions, sight distances, geometric considerations, coordinated systems and isolated signals, signal timing parameters, advanced detector locations, driver age, and turning movements.

Under NCHRP Project 03-95, Vanasse Hangen Brustlin, Inc., undertook the development of a comprehensive and uniform set of proposed guidelines for determining safe and operationally efficient yellow change and all-red clearance intervals at signalized intersections.

Initially they (1) reviewed and compared the definitions of yellow change and all-red clearance intervals, (2) conducted a critical review of relevant available literature, (3) conducted a survey of yellow and all-red timing practices at public agencies, and (4) reviewed past studies and agency operational experiences in relating change interval timing to red-light running and crashes. This information was synthesized to identify key stakeholder groups, point out knowledge gaps encountered in the research, and produce a draft outline of the guidelines. Field studies were conducted on critical factors including reaction time, deceleration rates, and the impact of the other factors identified as important in the design of change intervals. This information was analyzed to develop draft yellow change and all-red clearance interval guidelines, which were submitted to NCHRP and the key stakeholder groups for review and comment. Comments were addressed to produce a final report to document the research effort and the stand-alone, proposed guidelines for timing yellow and all-red intervals.

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Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

S U M M A R Y

Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections

For over 70 years, the subject of yellow and red signal indications has been a popular topic among scholars and professionals in the traffic engineering field. There has been much discussion throughout the industry and those associated with it about how to address the numerous issues and factors involved with developing change interval timing procedures that provide intersection safety while maintaining an acceptable level of operational efficiency. No consensus, however, has been reached to achieve this goal. The lack of a national standard, recommended practice, or set of guidelines for determining the duration of the yellow change and red clearance intervals has left each agency responsible for the timing of traffic signals to defend its own practices. Therefore, the objective of this research was to develop a comprehensive and uniform set of recommended guidelines for determining safe and operationally efficient yellow change and red clearance intervals at signalized intersections.

The yellow signal indication warns vehicle traffic of an impending change in right-of-way. It is displayed following every green signal indication. The amount of time that the yellow signal is displayed is referred to as the yellow change interval. The duration of this interval is based on the driver's perception-reaction time and deceleration rate, the approach speed, and the approach grade. The duration of the yellow change interval should allow, at a minimum, for a driver to comfortably decelerate to a stop prior to entering the intersection.

In many jurisdictions, the yellow change interval is followed by a red clearance interval. During the red clearance interval, a red signal indication is displayed to most (if not all) vehicular traffic approaches. The duration of the red clearance interval is based on intersection width, vehicle length, and the speed at which the vehicle traverses the intersection. The duration of the red clearance interval allows additional time as a safety factor for a driver that legally entered the intersection at the very last instant of the yellow change interval to avoid conflict with traffic releasing from an adjacent opposing intersection approach.

Over time, several national publications have served as references for the timing of change intervals. These include the *Manual on Uniform Traffic Control Devices,* the *Traffic Engineering Handbook,* the *Manual of Traffic Signal Design,* the *Traffic Control Devices Handbook,* and the *Traffic Signal Timing Manual.* A review of these publications indicated that, while each recommends using a consistent method to determine the yellow change and red clearance interval durations, each contains differing terminology and guidance. To further understand the "state of knowledge" of change interval timing methodologies and factors affecting these intervals, a review of previously published literature was conducted. Similarly, to identify commonalities and differences in the "state of practice" of change interval timing, a survey was distributed electronically to the national and international traffic engineering community. The findings of the "state of knowledge" and the "state of practice" confirmed that change interval timing practices, procedures, and considerations vary widely.

Fundamental driver behavior studies related to characteristics influencing the yellow change and red clearance intervals were performed more than 25 years ago. To verify that accurate values were being used in the timing of these intervals, an extensive field investigation of driver behaviors was conducted. The field investigation focused on three parameters directly related to the use of the kinematic equation: (1) perception-reaction time, (2) deceleration rate, and (3) approach speed.

The field investigation captured data from over 80 intersection approaches in five different states. The states were selected in different regions of the country to provide the regional diversity necessary to account for variations in timing practices as well as factors such as driver age, familiarity, grade, and red-light camera enforcement. Using a high-definition camera mounted atop a modular 20-foot-tall aluminum pole, over 320 hours of video were obtained. From this video, approximately 7,500 vehicles were extracted as data points for the evaluation portion of this study.

The analyses were performed using a multi-factor analysis of variance (ANOVA). The independent variables included both those related to the characteristics of the location and the site sampling variables. The dependent variables included perception-reaction time for first-to-stop vehicles, deceleration rate for first-to-stop vehicles, and approach speeds for both through vehicles and left-turn vehicles. The results of the analyses led to the following conclusions:

- The perception-reaction time was confirmed to be 1.0 s.
- The deceleration rate was confirmed to be 10 ft/s^2.
- The 85th percentile approach speed for through vehicles is closely approximated by adding 7 mph to the posted speed limit. The actual 85th percentile approach speed should be used in the kinematic equation; however, if field data are not available, this estimation is acceptable.
- The 85th percentile approach left-turn speed is closely approximated by subtracting 5 mph from the posted speed limit. This estimation should be used to calculate the yellow change interval. For red clearance interval calculations, the left-turn speed should be considered as 20 mph, regardless of the posted speed limit.

The analysis also examined the start-up delay exhibited by drivers on adjacent opposing intersection approaches. The results of the analysis revealed that initial start-up delay after the onset of green was approximately 1.1 seconds. The total time for an opposing vehicle to reach the nearest conflict point in the intersection was approximately 4.1 seconds. These findings supported the conclusion that a 1-second reduction can be applied to the calculated duration of the red clearance interval.

Consideration was given to the measurement of intersection width. The study determined that the intersection width should be measured from the back edge of the approaching movement stop line to the farthest edge of the farthest conflicting traffic lane. A pedestrian crossing equipped with pedestrian signals on a receiving lane should not be considered unless the nearest crossing line is 40 feet or more from the extension of the farthest edge of the farthest conflicting traffic lane. If this condition exists, the intersection width should be measured from the back edge of the approaching movement stop line to the nearest pedestrian crossing line. For left-turning traffic, the width of the intersection should be measured as the length of the approaching vehicle turning path from the back edge of the approaching movement stop line to the farthest edge of the farthest conflicting traffic lane, while also considering the presence and location of a pedestrian crossing equipped with pedestrian signals.

Lastly, the appropriate length of vehicle was examined. The research found no need to recommend a vehicle length different than the 20 feet currently being used in practice.

In conclusion, the duration of the yellow change and red clearance intervals has an impact on driver behavior and intersection safety. The survey results and the review of published literature confirmed that agencies responsible for change interval timing take a widely varied approach in their practices. It appears, however, that the kinematic equation (or a variation thereof) is used by most agencies and is commonly referred to in national publications used by the traffic engineering community. Therefore, the proposed guideline is based on the kinematic equation. The variables making up the kinematic equation have an effect on the resulting interval durations, particularly perception-reaction time and deceleration rate. The other variables to be considered when using the kinematic equation are approach speed, vehicle length, intersection width, and approach grade. Based on the results of this study, a recommended guideline for the timing of yellow change and red clearance intervals at signalized intersections has been proposed.

Assuming there is agreement with and acceptance of the guidelines for timing of the yellow change and red clearance intervals by the traffic engineering community, there does not appear to be any justification for additional research into this issue, specifically the formulation of the equation and its associated parameter values. However, it is suggested that further research of the safety impacts associated with implementing a red clearance interval be conducted. Given the concern of the need for a red clearance interval, it is recommended that research be conducted to isolate how the provision of a red clearance interval (and its length) affects the safety performance of the intersection.

CHAPTER 1

Introduction

Research Need

The function of a traffic signal is to alternate the right-of-way among the various traffic movements. This must be accomplished safely while minimizing intersection user delay and maximizing intersection capacity. Vital to accomplishing these goals is the yellow change interval. The yellow signal indication warns vehicle traffic that the green signal indication is being terminated and that a red signal indication will be exhibited immediately thereafter. Drivers approaching the intersection upon the display of a yellow signal must decide to stop or continue through the intersection. The duration of the yellow display is based on the driver's perception-reaction time (PRT) to the onset of the yellow and the distance needed to either safely stop or legally enter the intersection.

In many jurisdictions, the yellow change interval is followed by a red clearance interval (also referred to as "all-red"). During the red clearance interval, the red signal indication is displayed to all potentially conflicting traffic movements. It provides additional time as a safety measure to any driver that may have entered the intersection legally during the yellow change interval to avoid conflict with traffic releasing from an adjacent opposing intersection approach.

The yellow and red intervals are often referred to collectively as the "vehicle clearance interval" or "change interval." The selection of an appropriate yellow change interval length and the decision whether or not to employ a red clearance interval are important to both safety and capacity. Inadequate durations of these intervals may not provide an acceptable level of safety whereas unnecessarily long durations are counterproductive to efficient intersection operations. When implementing yellow change and red clearance intervals, there is a trade-off between intersection safety and intersection operations.

Red-light running (i.e., entering the intersection when the signal is red) is one of the most common causes of intersection crashes. Change interval duration is a significant factor affecting the frequency of red-light running, yet there remains no national consensus on how the yellow change and red clearance intervals should be timed for safe and efficient operations. The determination of the yellow change and red clearance intervals has come under scrutiny in recent years with the use of automated red-light running enforcement. Claims have been made that the yellow change intervals are too short, which induces red-light running, and results in higher numbers of citations. Studies of driver reaction times and vehicle deceleration rates used in determining appropriate yellow change and red clearance intervals were conducted more than 25 years ago. This research was based on the premise that these parameters needed to be addressed and that other important factors in calculating change intervals ought to be considered, including speeds, grades, vehicle types and mix, road surface conditions, geometry, coordinated systems and isolated signals, signal timing parameters, driver age, and turning movements.

Objective and Research Approach

The lack of a national standard, recommended practice, or set of guidelines for determining the duration of the yellow change and red clearance intervals has widened the scrutiny directed toward engineers and has left each agency responsible for the timing of traffic signals to defend its own practices. Therefore, the objective of this research was to develop a comprehensive and uniform set of recommended guidelines for determining safe and operationally efficient yellow change and red clearance intervals at signalized intersections. This objective was accomplished through the following activities:

- Developing a comprehensive inventory of factors to be considered in policies and procedures for calculating and implementing yellow change and red clearance intervals.
- Critically reviewing and summarizing available previously published literature relevant to factors potentially affecting change intervals and rational approaches to calculating yellow change and red clearance intervals.

- Surveying public agency yellow and red timing practices.
- Reviewing past studies and agency operational experiences relating change interval timing to red-light running crashes.
- Preparing an Interim Report that documented the findings of the above activities and providing a plan for the collection and analysis of relevant data associated with driver behavior in response to yellow signal displays.
- Conducting field studies that amassed data on critical factors, including PRT and deceleration rates.
- Synthesizing the field study results along with other information to formulate a recommended guideline for determining yellow change and red clearance intervals.

Several of the tasks required to complete this objective overlapped with concurrent research efforts for the Institute of Transportation Engineers (ITE) "Traffic Signal Change Intervals Recommended Practice" project.

Report Contents

The body of this report has been structured into the following additional chapters:

- Chapter 2 provides a general discussion of yellow change and red clearance intervals, the dilemma zone, and any existing standards or guidance provided in the Federal Highway Administration (FHWA) *Manual on Uniform Traffic Control Devices (MUTCD)* (*1*) or other national publications, especially those by the ITE.
- Chapter 3 summarizes the findings from the literature review on safety and operational issues relevant to yellow change and red clearance intervals.
- Chapter 4 documents the "state of practice" of agencies as determined from a survey and from the literature.
- Chapter 5 presents the methodology and results of the field studies.
- Chapter 6 provides a discussion of the findings from the previous chapters and how they are used to formulate the proposed guidelines.
- Chapter 7 presents the research conclusions and a suggestion for further research.
- Appendix A provides the proposed guideline for timing of yellow change and red clearance intervals.
- Appendix B provides sections of the *MUTCD* relevant to the meaning of vehicular signal indications, including the yellow change and red clearance intervals.
- Appendix C provides the definition of the yellow law as prescribed by each state's vehicle code.
- Appendix D provides a copy of the survey used to gather the information presented in Chapter 4.
- Appendix E provides detailed site characteristics for each location at which data were collected to generate the results in Chapter 5.
- Appendix F provides an analysis of the effect on intersection clearance of reducing the calculated red clearance interval by 1 second.

CHAPTER 2

Background

This chapter provides a discussion of the basic principles of yellow change and red clearance intervals and how they have been determined in earlier and current guidelines and manuals.

The change interval can consist of two components: the yellow change interval and, if used, the red clearance interval. The provision of these intervals is intended to provide a safe transition between conflicting vehicular movements. The yellow change interval warns vehicle traffic that the green signal indication is being terminated and that a red signal indication will be exhibited immediately thereafter. During the red clearance interval, a red signal continues to be displayed to all potentially conflicting vehicle traffic movements, providing additional time as a safety measure for vehicles that legally entered the intersection during the yellow change interval to avoid conflict with traffic releasing from an adjacent opposing intersection approach.

The duration of the yellow change and red clearance intervals has an impact on driver behavior and intersection safety. In addition, change interval duration reduces the amount of available green time in a cycle, therefore decreasing the capacity of the intersection. The goal of traffic engineers has been to find an optimum duration for the yellow change and red clearance intervals that improves intersection safety while maximizing intersection operations and efficiency.

The Dilemma Zone

The "dilemma zone" is a theoretical area of an intersection approach where a driver is presented with a condition (yellow signal indication) and a decision (stop or go). One of the earliest references to the dilemma zone was by Gazis et al. (*2*). This work recognized that a driver presented with an inadequately timed yellow signal indication may be traveling at a speed such that he/she is too close to the intersection to comfortably stop yet too far away to completely clear the intersection prior to the onset of the conflicting green signal indication. This has been referred to as the Type I dilemma zone. Figure 1 illustrates the Type I dilemma zone concept.

Urbanik and Koonce (*3*) noted that the work by Gazis et al. was consistent with the ITE recommended practice of the time; however, they stated that the ITE formula had since been modified, providing equations for the yellow change interval and the red clearance interval. The ITE equation provides adequate time for drivers traveling at or below the assumed approach speed to reach the intersection during the yellow change interval and to clear the intersection during the red clearance interval.

Traffic engineers and practitioners have been concerned with providing an adequate duration for the yellow change and red clearance intervals such that the Type I dilemma zone does not occur, while also considering operational inefficiencies incurred by providing excessive durations for these intervals. Under the traditional definition of a Type I dilemma zone, a well-timed change and clearance interval (yellow + red) allows for drivers farther away from the intersection to decelerate comfortably to a stop and for drivers closer to the intersection to safely continue to the far side. Conversely, a change interval of insufficient duration could expose drivers legally entering the intersection to conflict with opposing traffic at the onset of the green signal indication. The traditional Type I dilemma zone definition is conservative based on the premise that the intersection must be clear of all potential conflicts prior to the release of opposing traffic. However, as discussed later in this report, "intersection clearance" can be redefined to account for vehicular start-up delay, spatial buffers, and driver legal operation obligations, without having negative safety consequences.

In 1974, the "option zone" or "indecision zone" was identified in a report by the Southern Section ITE Technical Committee (*4*). This has been referred to as the Type II dilemma zone and occurs as a result of different drivers displaying indecision about whether to stop or go when presented

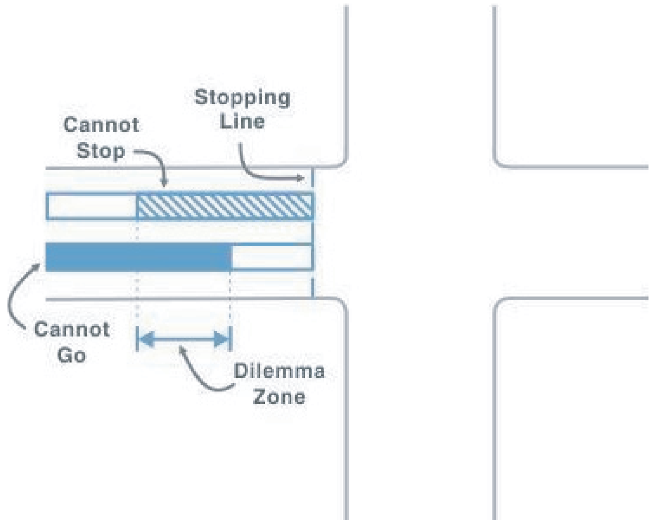

Figure 1. Type I dilemma zone concept (3).

the yellow signal indication. Figure 2 illustrates the Type II dilemma zone concept.

The ITE Technical Committee defined the boundaries of this zone as the distance interval in which driver stopping probability was between 10 and 90 percent. Since then, researchers have attempted to identify definitive distance or time boundaries of this zone. Zegeer and Deen (*5*) based their research on distance from the stop line, plotting stopping probability curves for five different speed limits and relating them to the distance of a vehicle from the intersection. As concluded by Bonneson et al. (*6*), most time-based studies have indicated the boundaries of a Type II dilemma zone to be between 2.5 and 5.5 seconds from the stop line, corresponding approximately to the 10 percent and 90 percent boundaries originally stated by the ITE Technical Committee. The *Traffic Control Systems Handbook* (*7*) acknowledges the upper boundary (90th percentile stopping probability) as approximately 4.5 to 5.0 seconds from the stop line and the lower boundary (10th percentile stopping probability) as approximately 2.0 to 2.5 seconds from the stop line.

Type II dilemma zones will continue to exist at the onset of every yellow indication, regardless of change interval durations. This is due to the fact that drivers will react differently when facing a yellow signal indication, regardless of whether adequate yellow time is provided, based on prevailing conditions.

National Resource Publications

Over time, several national publications have served as references for the timing of change intervals, including:

- FHWA *MUTCD* (*1*),
- ITE *Traffic Engineering Handbook* (*8*),
- ITE *Manual of Traffic Signal Design* (*9*),
- ITE *Traffic Control Devices Handbook* (*10*),
- FHWA *Signalized Intersections: Informational Guide* (*11*),
- FHWA *Traffic Signal Timing Manual* (*12*), and
- FHWA *Yellow Change Intervals Memorandum* (*13*).

The current edition of the *MUTCD* (2009) directs practitioners to the ITE *Manual of Traffic Signal Design* or the ITE *Traffic Control Devices Handbook* for engineering practices to determine the durations of the yellow change and red clearance intervals. All other national publications by

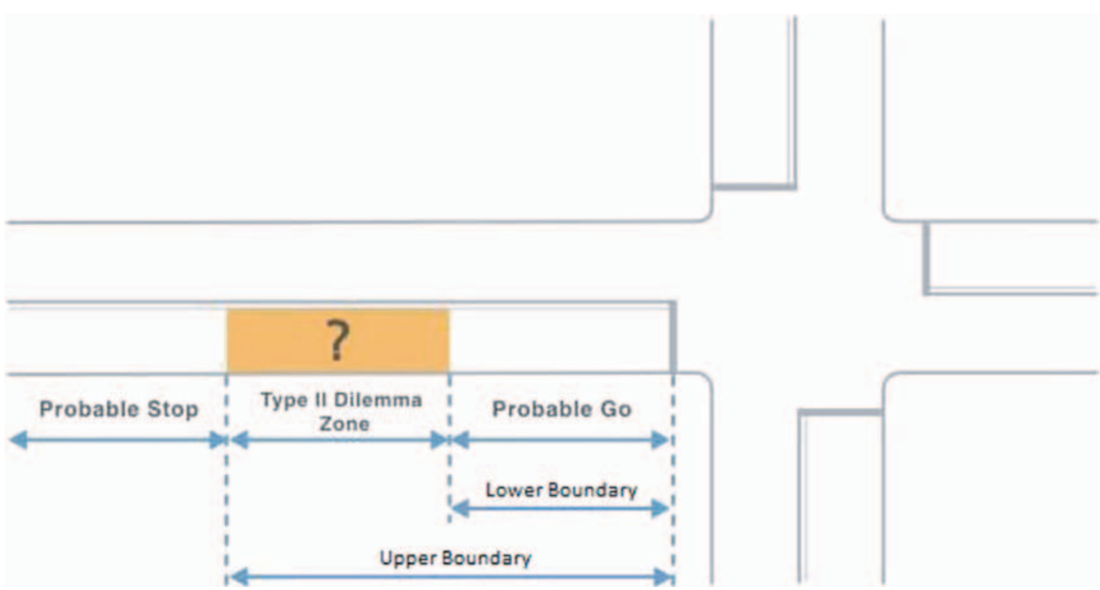

Figure 2. Type II dilemma zone concept (3).

ITE and FHWA suggest using the equations provided in the ITE *Traffic Engineering Handbook.* While each publication recommends using a consistent method (i.e., the kinematic equation), each contains different terminology and guidance.

Manual on Uniform Traffic Control Devices (MUTCD)

The *MUTCD* (*1*) is the primary document for traffic control device standards, including traffic signalization. The intent of the *MUTCD* is to establish uniformity for all traffic control devices so that users encounter consistent applications across the nation. Each state annually certifies compliance with the *MUTCD;* therefore, all states should be using the same meaning of signal indications. Sections of the *MUTCD* relevant to the meaning of vehicular signal indications, including the yellow change and red clearance intervals, are presented in Appendix B. Some of the key paragraphs concerning the timing of the yellow change and red clearance intervals are as follows:

> **Section 4D.26 Yellow Change and Red Clearance Intervals**
> . . .
> 03 **The duration of the yellow change interval shall be determined using engineering practices.**
> . . .
> *Guidance:*
> 05 *When indicated by the application of engineering practices, the yellow change interval should be followed by a red clearance interval to provide additional time before conflicting traffic movements, including pedestrians, are released.*
> **Standard:**
> 06 **When used, the duration of the red clearance interval shall be determined using engineering practices.**
> Support:
> 07 Engineering practices for determining the duration of yellow change and red clearance intervals can be found in ITE's "Traffic Control Devices Handbook" and in ITE's "Manual of Traffic Signal Design" (see Section 1A.11).
> . . .
> *Guidance:*
> 14 *A yellow change interval should have a minimum duration of 3 seconds and a maximum duration of 6 seconds. The longer intervals should be reserved for use on approaches with higher speeds.*
> 15 *Except when clearing a one-lane, two-way facility (see Section 4H.02) or when clearing an exceptionally wide intersection, a red clearance interval should have a duration not exceeding 6 seconds.*

Institute of Transportation Engineers (ITE)

Beyond the *MUTCD,* traffic engineers look to ITE for guidance on traffic engineering issues. In 1985, ITE developed a proposed recommended practice based on the following kinematic equation for calculating the "vehicle clearance interval" or "change period" (*14*):

$$CP = t + \frac{V}{2a + 64.4g} + \frac{W + L}{V}$$ **Equation 1**

Where:
- CP = change period (s);
- t = PRT (usually 1 s);
- V = approach speed (ft/s);
- a = deceleration rate (ft/s^2);
- g = percent of grade divided by 100 (plus for upgrade, minus for downgrade);
- W = width of intersection (ft); and
- L = length of vehicle (ft).

[To maintain consistency, the symbols and terminology might be different than used in respective referenced publications.]

Although ITE never formally adopted this proposed recommended practice, many agencies use it today (as discussed in Chapter 4). The equation provides time for a driver to perceive and react to the yellow indication, and either decelerate comfortably to a stop or continue through the intersection prior to a change in right-of-way. Generally, the first two terms of this equation are used to calculate the yellow change interval, while the third term is used to calculate the red clearance interval.

According to the current ITE *Traffic Engineering Handbook* (*8*), the yellow change interval is calculated as follows:

$$Y = t + \frac{V}{2a + 2Gg}$$ **Equation 2**

Where:
- Y = yellow interval (s);
- t = reaction time (typically 1 s);
- V = design speed (ft/s);
- a = deceleration rate (typically 10 ft/s^2);
- G = acceleration due to gravity (32.2 ft/s^2); and
- g = grade of approach (percent / 100, downhill is negative grade).

The current ITE *Traffic Engineering Handbook* only provides one equation for calculating the red clearance interval. The following is an excerpt from the discussion of the red clearance interval:

> *The Red Clearance Interval is an optional interval that follows a yellow change interval and precedes the next conflicting green interval. The red clearance interval is used to provide additional time following the yellow change interval before conflicting traffic is released. The appropriate red time for the approach should be calculated using the formula found in ITE's Determining Vehicle Signal Change and Clearance Intervals:*
>
> $$R = \frac{W + L}{V}$$ **Equation 3**

Where:
R = red interval (s),
V = design speed (ft/s),
W = width of stop line to far-side no-conflict point (ft), and
L = length of vehicle, typically 20 ft.

For exclusive turn movements, the value of W should be measured along the vehicle turn path from the stop line to the no-conflict point.

The decision to use a red clearance interval is determined by intersection geometrics, crash experience, pedestrian activity, approach speeds, local practices, and engineering judgment.

It is important to note that older versions of the ITE *Traffic Engineering Handbook* (*15*) provide three equations for calculating the red clearance interval based on various pedestrian conditions:

1. Where there is no pedestrian traffic,

$$R = \frac{W + L}{V} \qquad \textbf{Equation 4a}$$

This equation is intended to place the vehicle entirely out of the area of conflict with vehicular traffic that is about to receive a green indication.

2. Where there is the probability of pedestrian crossings,

$$R = \frac{P}{V} \qquad \textbf{Equation 4b}$$

This equation is intended to place the vehicle at a point directly in front of pedestrians waiting to cross the far-side crosswalk.

3. Where there is significant pedestrian traffic or pedestrian signals protect the crosswalk,

$$R = \frac{P + L}{V} \qquad \textbf{Equation 4c}$$

This equation is intended to place the vehicle entirely out of the area of conflict with pedestrians crossing the far-side crosswalk and vehicular traffic that is about to receive a green indication.

Where:
R = length of red clearance interval, to the nearest 0.1 s;
W = width of the intersection, in ft, measured from the near-side stop line to the far edge of the conflicting traffic lane along the actual vehicle path;
P = width of the intersection, in ft, measured from the near-side stop line to the far side of the farthest conflicting pedestrian crosswalk along the actual vehicle path;
L = length of vehicle, recommended as 20 ft; and
V = speed of the vehicle through the intersection, in ft/s.

A few noteworthy points about the equations presented here and the information provided in the national publications reviewed for this research:

- The *Traffic Engineering Handbook* (*8*) does not specify the approach speed measure to be used (i.e., posted speed limit, 85th percentile speed, etc.). Instead, "design speed" is referenced without any further explanation. It does state that consideration should be given to the 15th percentile speed, particularly at wider intersections. All other sources recommend using the 85th percentile approach speed. If unknown, the approach speed should be substituted with the approach speed limit. However, the *Yellow Change Intervals Memorandum* (*13*) states that, in the absence of the 85th percentile approach speed, the approach speed limit plus 10 mph should be used.
- The *Traffic Control Devices Handbook* (*10*):
  - Defines the width of the intersection (W) as the full intersection width plus the crosswalk widths.
  - Assumes vehicle length (L) to be 15 feet rather than 20 feet.
  - Provides a modified red clearance interval equation that accounts for one (1.0) second of reaction time delay (Equation 5). This equation provides partial clearance of the intersection before the onset of green for conflicting movements. The rationale behind use of this equation is that the conflicting traffic experiences delay in starting due to reaction time and is some distance away from a conflict point.

$$R = \frac{W + L}{V} - 1 \qquad \textbf{Equation 5}$$

Where:
R = red clearance interval, s;
W = width of the intersection from stop line to end of the far-side crosswalk, ft;
L = length of vehicle, feet (use 15 ft); and
V = 85th percentile speed, ft/s.

- The *Manual of Traffic Signal Design* (*9*) and the *Signalized Intersections: Informational Guide* (*11*) present only the single standard equation approach rather than separate equations for the yellow change and red clearance intervals. However, the *Manual of Traffic Signal Design* does acknowledge that many agencies set the yellow change interval equal to the first two terms of the equation. Likewise, the *Guide* notes that some agencies use the third term of the equation to calculate the duration of the red interval.

- The *Traffic Signal Timing Manual* (*12*) presents only the single standard equation approach rather than separate equations for the yellow change and red clearance intervals. The discussion reiterates that the use of a red clearance interval is optional, and that there is no consensus regarding its application or duration. The manual notes that the third term of the single standard equation is to be used to determine the duration of the red interval; however, it is not prescriptive with regard to values for intersection width, vehicle length, or speed, citing "local policy" or "engineering judgment."

Vehicle Code and Timing of Yellow Change and Red Clearance Intervals

Each state determines the law regarding the entry of vehicles into the intersection during the yellow change interval. The laws pertaining to the yellow change interval may be categorized into either "permissive" entry or "restrictive" entry. Under a "permissive" yellow law, drivers may enter the intersection during the entire duration of the yellow change interval and legally be in the intersection while the red signal indication is displayed, so long as entrance occurred before or during the yellow signal indication. The red clearance interval provides additional time as an added safety factor prior to a change in right-of-way. Per the *MUTCD* (*1*), drivers presented with a green signal indication on an adjacent opposing intersection approach are obligated to yield the right-of-way to other drivers that have legally entered the intersection. Under the "restrictive" yellow law, (1) drivers may not enter the intersection during the yellow signal indication unless it can be entirely cleared prior to the onset of the red signal indication, or (2) drivers may not enter the intersection unless it is impossible or unsafe to stop. States following the second condition of the "restrictive" yellow law are generally not in conflict with the "permissive" yellow law.

The timing of the yellow change interval will ideally concur with the law. This becomes important when using the ITE equation to calculate the duration of the yellow and red signal indications, particularly with respect to allocation of the change and clearance times. In "permissive" applications, the first two terms of the ITE equation are typically used to calculate the yellow interval and applied to provide only enough yellow time for vehicles to enter the intersection, but not necessarily to clear to the far side of the intersection. The necessary clearance time is the red interval, calculated using the third equation term. In "restrictive" applications, all three terms of the ITE equation are typically used to calculate the yellow interval. A "restrictive" yellow law does not necessitate the use of a red clearance interval, but does rely on the use of good engineering judgment (*2*).

The definitions of all 50 states' vehicle codes pertaining to the yellow change interval are included in Appendix C. Those states that do not address this in their vehicle code (i.e., Alaska and Massachusetts) defer to the *Uniform Vehicle Code* (*16*). The *Uniform Vehicle Code* is a set of traffic laws prepared by the National Committee on Uniform Traffic Laws and Ordinances (NCUTLO). The *Code* specifies a "permissive" yellow law; therefore, those states deferring to the *Code* follow the "permissive" yellow law by default. Four states follow a true "restrictive" yellow law: Louisiana, Rhode Island, Tennessee, and West Virginia. Of the remaining states, 37 follow a "permissive" yellow law and nine follow the second condition of the "restrictive" yellow law.

CHAPTER 3

State of Knowledge for Change Interval Timings

This chapter presents the summary findings from a review of available previously published literature to understand the state of knowledge for change interval timing and the associated factors that affect how yellow change and red clearance intervals are determined and implemented. The review summarizes research related to interval calculations, PRTs, deceleration rates, approach speeds, and any other factors that may affect the timing of yellow change and red clearance intervals. The chapter also contains a review of prior studies and agency experiences that relate change interval timing practices to red-light running behavior and crash experience. The detailed literature review was presented as an appendix in the Interim Report for this research; it can be obtained from NCHRP.

Change Interval Timing Procedures and Parameters

Determination of Change Intervals

While most agencies use the ITE kinematic equation (as seen from the survey results presented in Chapter 4), and national publications direct the traffic engineering community to this method, there are several other methods used for calculating the yellow change and red clearance intervals. Most methods can be classified as one of the following:

- Kinematic Equation—The method calculates change interval durations based on a formula that includes PRT, vehicle speed, vehicle deceleration rate, approach grade, acceleration due to gravity, intersection width, and vehicle length. There are several variations of this method, most relating to the allocation of time between the yellow change and red clearance intervals. The kinematic equation provides the basis for the ITE equation (*14*).
- "Rule-of-Thumb"—Some engineers and practitioners use the approach speed in miles per hour divided by 10 to determine the length of the yellow change interval. The 85th percentile speed or the posted speed limit is used as the approach speed. This results in common lengths of 3 to 5 seconds for the yellow change interval.
- Uniform Value—Some jurisdictions use a uniform yellow change interval length for all of the intersections. The traffic engineer decides this yellow change interval length based upon local conditions.
- Uniform Value by Speed—A variation of the Uniform Value method is to use a set of uniform values based on the posted speed limits, instead of one uniform value for the whole system.

A study by Tarnoff (*17*) confirmed the variation of change interval timing practices across the United States. A state-of-practice survey showed that yellow change interval calculations included the kinematic-based formula, the rule-of-thumb method in which intervals varied with speed, and the application of uniform intervals for a given area. Likewise, red change interval calculations included the kinematic-based formula and the application of uniform intervals. Those agencies using the kinematic-based formula generally applied all three terms of the equation to the yellow change interval in the absence of a red clearance interval; red clearance interval calculations applied the third term of the equation only. Of the agencies surveyed, 24 percent used a single value of yellow time for intersections with similar characteristics; 27 percent used a single value of red time in the same manner.

A state-of-practice survey conducted in British Columbia, Canada, by Voss (*18*) revealed that change interval timing practices are just as variable outside of the United States. Respondents indicated that change interval timing methods included the ITE equation, the rule-of-thumb method based on approach speeds, and the application of a constant value.

All methods proposed by ITE for determining change intervals in the past 70 years have been based on the kinematic model. *A History of the Yellow and All-Red Intervals for*

Traffic Signals (*19*) provides a comprehensive review of these models and formative research prior to the kinematic model. The report indicates that the standard kinematic model has had few changes since its adoption in 1965. A modification factor to accommodate approach grade was incorporated in 1982 and has since been in the proposed method. Current literature presents the following methods for determining the yellow change intervals:

- Kinematic equation method,
- "Rule-of-thumb" method,
- Uniform value method,
- Stopping probability method,
- Combined kinematic model and stopping probability method, and
- Modified kinematic model for left-turn movements.

A History of the Yellow and All-Red Intervals for Traffic Signals also provides a comprehensive review of the history of the red clearance interval. The report references the 1950 edition of ITE *Traffic Engineering Handbook* as the first mention of the red clearance interval and the 1985 *Determining Vehicle Change Intervals: A Proposed Recommended Practice* for proposing a separate red clearance interval calculation using one of three kinematic model-based equations. The current literature presents the following methods for determining the red clearance interval:

- Kinematic equation method,
- Uniform value method,
- Conflict zone method, and
- Modified kinematic model for left-turn movements.

Perception-Reaction Time

PRT refers to the time needed for an approaching driver to "perceive" the yellow indication and to "react" to the indication by braking to a stop or deciding to pass through the intersection. When used in change interval calculations, this variable takes into account the delay in reaction time caused by human behavior.

The PRT variable has the second largest effect on the variance of the calculated change interval (*20*). A major difficulty in applying the PRT lies in determining an appropriate value that is representative of the driver population.

Since 1965, the ITE *Traffic Engineering Handbook* has consistently suggested using 1 second for the PRT in calculating the yellow change interval. According to a 1983 FHWA publication (*21*), this value is based on a 1934 Massachusetts Institute of Technology study on braking. In 1985, an ITE technical council evaluated PRT values and deemed the 1 second value appropriate. Recent studies also confirmed many previous findings that support the practice of using a 1 second PRT (*22, 23*).

The current literature and "state of practice" do not support using a value greater than 1 second for average PRT. Regarding the accommodation of older drivers, the *Highway Design Handbook for Older Drivers and Pedestrians* (*24*) notes that 1 second is sufficient PRT.

Approach Speed

A History of the Yellow and All-Red Intervals for Traffic Signals (*19*) provides a comprehensive history of the approach speed variable used in change interval calculations. According to the report, the 85th percentile speed is commonly used today, although the recommended value has changed over the past 60 years.

The ITE *Determining Vehicle Change Intervals: A Proposed Recommended Practice* (*14*) states that the 85th percentile speed is most representative of the approach speed, but additionally notes that the posted speed limit may be preferred to avoid extensive field work. The report also suggests that different approach speeds may be appropriate for calculating the yellow change and red clearance intervals.

The current ITE *Traffic Engineering Handbook* (*8*) specifies using "design speed" in the calculation of the yellow change interval. It also suggests giving consideration to using the 15th percentile speed when calculating yellow clearance time, especially at wider intersections. However, this statement would apply to the red clearance time and would not be applicable to the yellow change interval.

NCHRP Report 504: Design Speed, Operating Speed, and Posted Speed Practices (*25*) reported a strong relationship between operating speed, or the 85th percentile speed, and the posted speed limit. The regression analysis revealed that the 85th percentile speed is approximately 7 miles per hour greater than the posted speed limit; however, this applied to roadway sections and not necessarily to intersection approaches.

Approach speeds for turning vehicles differ from through-movement vehicles. In many cases, left-turning drivers are already braking at the onset of the yellow change interval, thereby greatly reducing or eliminating the PRT in response to the yellow indication. Thus, traditional yellow timing based on the approach characteristics of through-moving vehicles has not been recommended for exclusive left-turn phases. Instead, methods such as those proposed by Yu et al. (*26*) are recommended. The Yu et al. study developed a method for determining the yellow change and red clearance intervals based on several site-related input variables, including the approach speed upstream of the intersection, speed at entry to the intersection, speed during the left-turn maneuver, trajectory of the left-turn path, percent of trucks, and other factors. The model was calibrated based on data from 21 Texas intersections. The study concluded that yellow change and red clearance intervals for left-turn phases are typically incorrect if they are computed based on through movements. The specific conclusions pertaining to the 21 Texas intersections were as follows:

- Existing yellow change intervals for left-turn phases were too long;
- Existing red clearance intervals for left-turn phases were too short; and,
- Existing total change intervals (yellow plus red) for left-turn phases were of the correct duration.

The Yu et al. study recommended the following yellow and red durations for left-turn phases:

- For locations with approach speeds of 50 mph and below:
  - Yellow Interval = 3.0 seconds, and
  - Red Interval = 2.2 seconds to 4.6 seconds (depending on speed and clearing distance).
- For locations with approach speeds of 55 mph and above:
  - Yellow Interval = 3.0 seconds to 3.4 seconds, and
  - Red Interval = 3.1 seconds to 4.5 seconds (depending on clearing distance).

As part of the development of a proposed change interval calculation method for left-turning vehicles, data confirmed that left-turn approach speeds are lower than through-movement approach speeds (*27*). The mean approach speed for left-turning vehicles was reported to range from 29.37 to 36.24 miles per hour. Other studies have reported 85th percentile speeds for left-turning vehicles to range between 15 and 25 miles per hour (*28, 29, 30*).

Grade

The approach grade variable modifies a vehicle's stopping ability based on the slope of the roadway at the approach to the intersection. *A History of the Yellow and All-Red Intervals for Traffic Signals* (*19*) references the 1982 edition of the *Manual of Traffic Signal Design* for the first inclusion of grade in calculating change intervals. The report suggests the consideration of grade may have been the result of work by Parsonson and Santiago (*31*). Subsequent ITE publications have included the approach grade variable in the kinematic equation calculation method.

The FHWA *Traffic Signal Timing Manual* (*12*) suggests that, for every 1 percent upgrade, the duration of the calculated yellow change interval decreases by 0.1 seconds. Conversely, for every 1 percent downgrade, the duration of the calculated yellow change interval increases by 0.1 seconds.

Deceleration Rate

The deceleration rate of an approaching vehicle has the largest effect on the variance of the calculated change interval (*20*). The most recent recommendations on deceleration rate suggest applying a value of 10 feet per second per second (ft/s^2), which is supported by several studies (*32, 33, 34, 35*).

The 1965 edition of the *Traffic Engineering Handbook* (*36*), however, suggested 15 ft/s^2 as a reasonable deceleration rate. The 1982 *Manual of Traffic Signal Design* (*9*) was modified and suggested applying a 10 ft/s^2 deceleration rate. All subsequent editions of the *Traffic Engineering Handbook* have suggested 10 ft/s^2.

The findings of a 2007 experimental study supported a significant relationship between deceleration rate and time to stop line to age. Deceleration rate decreased as drivers were farther from the stop line. The mean deceleration rate for 18- to 35-year-old drivers was 14.4 ft/s^2, compared to 12.5 ft/s^2 for 55- to 64-year-old drivers and 12.3 ft/s^2 for 65-year-old and older drivers (*22*).

The fifth edition of AASHTO's *A Policy on Geometric Design of Highways and Streets* (*37*) suggests an 11.2 ft/s^2 comfortable deceleration rate for calculating the stopping sight distance. No guidance is given on applying this value for calculating change intervals.

Width of Intersection

The variable for intersection width considers the distance that a vehicle must travel to clear the intersection or potential conflict zone within the intersection. Definitions and guidance on measuring this variable vary. Measurement of the intersection width may begin at the stop line or the near conflicting curb line. For through movements, the measurement may extend to the far conflicting curb line or farthest conflicting crosswalk line. Measuring intersection width for left-turn movements may involve measuring the curved vehicle path.

A History of the Yellow and All-Red Intervals for Traffic Signals (*19*) provides a summary of guidance on the width of the intersection in the past. The report notes that minimal guidance has been provided in past editions of the ITE *Traffic Engineering Handbook,* and suggests that guidance could be strengthened in the future. The current *Handbook* (*8*) states that, for exclusive turning movements, the intersection width should be measured along the vehicle path from the stop line to the no-conflict point.

Vehicle Length

The vehicle length variable takes into account the length of an average vehicle that must clear the intersection or conflict point. The 1965 edition of the *Traffic Engineering Handbook* (*36*) suggested a 20-foot vehicle length. Subsequent guidance by ITE agrees with this value. In 1977, Williams (*38*) suggested a 17-foot vehicle length for use in his combined kinematic model and stopping probability method.

The fifth edition of AASHTO's *A Policy on Geometric Design of Highways and Streets* (*37*) provides groupings of selected vehicles (i.e., design vehicles) used to establish highway design controls. According to this publication, the length of a passenger car design vehicle is 19 feet. AASHTO also suggests that the WB-65 or WB-67 be the minimum size design truck for the geometric design of intersections on state highways and industrialized streets that carry high volumes of truck traffic and/or that provide local access for large trucks. The length of a WB-65 or WB-67 design vehicle is 73.5 ft. The use of a WB-50 is typically used for intersection design. This design vehicle has a length of 55 ft.

Effects of Change Intervals on Driver Behavior

Recent studies have explored the factors that may influence driver behavior in response to change intervals. These factors include driver, vehicle, and environmental characteristics. Driver characteristics may consist of age, gender, and experience. Vehicle characteristics may include condition, type, or model. Environmental characteristics consider other external factors such as weather condition, time of day, traffic volume, road classification, number of lanes, surrounding land use, regional driving practices, and level or type of enforcement.

A 2005 study for FHWA asked focus group and survey participants how they would react to hypothetical traffic situations (*39*). The participants included 18- to 35-year-old, 35- to 55-year-old, and 65-year-old and older drivers of both genders from Washington, D.C., Chicago, and Seattle. Their stated preferences indicated that older drivers were more likely to stop at the yellow indication to avoid running a red light, while middle-aged and younger drivers would run the red light. The results also showed that driver behavior is influenced by attitude, beliefs, and social norms.

The 2008 experimental study by Rakha et al. (*40*) observed driver behavior at change intervals in a testing facility. The researchers concluded that older drivers' dilemma zones had greater variance and were closer to the intersection than those of middle-aged and younger drivers. The findings additionally suggested that female drivers were more likely to stop at the intersection after the onset of the yellow indication and had dilemma zones closer to the intersection compared to male drivers.

Effects of Change Intervals on Safety

Numerous studies over the past 50 years have attempted to examine and quantify various safety effects associated with modifications to change interval timing and phasing. These studies generally fall into three categories: effects of change interval timing on red-light running and late exits, effects of change interval timing on crashes, and crash effects associated with installing red clearance intervals. There is a broad range in the quality of these studies, and consequently in the reliability of the results. This review attempted to identify all relevant and available reports, assess the quality of the studies, document references, and provide a synthesis of the methods and main results. In summary, and despite the diversity of research methods and range of findings, the following general conclusions can be drawn from the available body of literature.

Effects of Change Interval Timing on Red-Light Running and Late Exits

At intersection approaches where yellow signal timing duration is set below values associated with ITE guidelines or similar kinematic-based formulae, increasing yellow change interval duration to achieve ITE guidelines can significantly reduce red-light running. Studies by Bonneson and Zimmerman (*41*), Harders (*42*), Munro and Marshall (*43*), Retting et al. (*44*), van der Horst and Wilmink (*45*), and Wortman et al. (*46*) found that increasing yellow change interval duration by about 1 second at approaches that were deemed to have insufficient change interval timing was associated with reductions in red-light running ranging from about 36 to 90 percent. This range includes a number of weak study designs. The best estimate of effect on red-light running, based on better designed studies, is about 36 to 50 percent reduction. Likewise, increasing yellow change and/or red clearance interval timing to achieve values associated with ITE guidelines or similar kinematic-based formulae can significantly reduce late exits, as well as conservatively-defined potential vehicle conflicts. Evidence generally shows that increasing the duration of red intervals does not increase red-light running.

Effects of Change Interval Timing on Crashes

Prior studies report a range of crash effects associated with modifications to change interval timing, reflecting differences in research methods, outcome measures, settings, specific types of modification to change interval timing, and other factors. Several crash-based studies report that setting change interval timing to values associated with ITE guidelines is associated with reduced risk of total crashes, injury crashes, and/or right-angle crashes. The best estimate of effect on crashes, based on better designed before–after studies, is about an 8 to 14 percent reduction in total crashes, and about a 12 percent decrease in injury crashes. Some studies report evidence of increased risk of rear-end crashes when yellow change interval duration is increased, which may reflect the increased exposure of drivers to this decision period. Benioff et al. (*47*) concluded that excessively long yellow change intervals "definitely are hazardous."

Crash Effects Associated with Installing Red Clearance Intervals

This optional signal phase has many supporters in the traffic engineering community who believe the use of red intervals helps to prevent right-angle crashes associated with drivers that enter late in the yellow phase or who run red lights. Red intervals also have detractors who argue that they simply encourage and reward red-light running behavior. Unfortunately, crash-based research evaluations do not provide a clear indication of the safety effects of installing red intervals. Most available information comes from uncontrolled before–after experience or studies that suffer from relatively weak experimental designs. Results range from relatively large crash reductions to modest crash reductions to crash increases to no effects. The strongest study on this topic, though still with methodological limitations, was conducted by Souleyrette et al. (*48*) and suggests modest short-term crash reductions, but no long-term effects associated with installing red clearance intervals. Another study by Roper et al. (*49*) evaluating the effect of red clearance intervals also indicated no long-term safety benefit. Absent more definitive research, the crash effects of installing red intervals are unclear. Although a number of authoritative publications on crash reduction factors and intersection safety provide indications of crash reductions associated with installation of red intervals (e.g., FHWA's *Signalized Intersections Informational Guide;* FHWA's *Desktop Reference for Crash Reduction Factors;* Kentucky Transportation Center's *Kentucky Accident Reduction Factors;* Southeast Michigan Council of Governments' (SEMCOG) *Traffic Safety Manual*), the reported effectiveness is based on uncontrolled before–after experience or studies that suffer from relatively weak experimental designs.

State-of-Knowledge Summary

The following tables summarize the key findings of the literature review. Table 1 presents information relative to change interval timing procedures and parameters. Table 2 focuses on the effects of change intervals with regard to driver behavior and safety.

Table 1. Key findings of literature review for change interval timing procedures and parameters.

Issue	Findings
Determination of Yellow Change Interval Duration	-Variety of methods including Kinematic Equation, Rule-of-Thumb, Uniform Value, Stopping Probability, Combined Kinematic Equation and Stopping Probability, and Modified Kinematic Equation for Left-Turn Movements.
Determination of Red Clearance Interval Duration	-Variety of methods including Kinematic Equation, Uniform Value, Conflict Zone, and Modified Kinematic Equation for Left-Turn Movements.
PRT	-Second largest effect on variance of calculated change interval. -Current literature and state of practice do not support using an average value greater than 1 second.
Approach Speed	-85th percentile speed suggested as most representative of approach speed. -Posted speed limit may be preferred to avoid extensive field work. -Left-turning vehicle speeds are lower than through movement (e.g., 15–25 mph, upwards to 29.37–36.24 mph).
Deceleration Rate	-Largest effect on variance of calculated change interval. -Current literature and state of - practice suggests value of 10ft/s^2. -AASHTO suggests 11.2 ft/s^2 as a comfortable deceleration rate for stopping sight distance calculations.
Grade	-For every 1 percent upgrade, the duration of the calculated yellow change interval is decreased by 0.1 seconds. -For every 1 percent downgrade, the duration of the calculated yellow change interval is increased by 0.1 seconds.
Intersection Width	-Definitions and guidance vary. -Suggested that future guidance be strengthened.
Length of Vehicle	-Typically assumed as 20 ft. -AASHTO design vehicles: passenger car (19 ft), single-unit truck (30 ft), WB-50 (55 ft).

Table 2. Key findings of literature review for effects of change intervals on driver behavior and safety.

Issue	Findings
Effect of Change Intervals on Driver Behavior	-Behavior influenced by traffic speed and volume, signal timing and coordination, number of lanes (i.e., intersection width), vehicle type, age, and gender of driver. -Other factors include weather conditions, regional driving practices, level and/or type of enforcement, and cell phone use. -Studies have shown that female drivers and older drivers are more conservative than their male/younger counterparts.
Effect of Change Intervals on Safety	-Increasing the yellow change interval duration to ITE guidelines has been shown to reduce red-light running by 36 to 50 percent. -Increasing the red clearance interval duration to ITE guidelines generally does not increase the occurrence of red-light running. -Setting change interval timings to ITE guidelines has been shown to reduce total crashes by 8 to 14 percent and injury crashes by approximately 12 percent. -Studies show a possibility of an increase in rear-end crashes when yellow change interval durations are increased. -Crash effects of installing red clearance intervals at intersections previously without are unclear.

CHAPTER 4

State of Practice for Change Interval Timings

There is much discussion throughout the industry and those associated with it (i.e., government officials/lawmakers, law enforcement, etc.) about how to address the numerous issues and factors involved with developing change interval timing procedures. No consensus has been reached, however, as to how to achieve the most safe and operationally efficient change interval timings. This chapter presents the results of a survey distributed electronically to the traffic engineering community, nationally and internationally. Based on the responses received, it is apparent that there are many variations used in practice to determine change interval timings.

Survey of State Agencies

Previous Survey in 2008

In 2008, a review of yellow and red policies of 22 states was conducted for the Commonwealth of Virginia (*50*). The search included collecting various traffic engineering or signal design manuals and other documents on the state agency websites. Telephone or email surveys also supplemented the Internet search. Some of the most significant findings of the 2008 review are as follows:

- 20 of 22 states follow the equation in the ITE *Traffic Engineering Handbook* for calculating the yellow change interval (i.e., allocate the first two terms). However, many states use different values for the parameters of PRT, approach speed, and deceleration rate. The two other states used the "rule-of-thumb" method.
- 16 of 22 states follow the first equation in the ITE *Traffic Engineering Handbook* for calculating the red clearance interval (i.e., allocate the third term only), although definitions of intersection width differed. Other methods include using the second equation of the ITE *Traffic Engineering Handbook* for calculating the red clearance interval, a modified version of the first equation of the ITE *Traffic Engineering Handbook* for calculating the red clearance interval, and the "rule-of-thumb" method.
- 19 of 20 states following the ITE equation use a PRT of 1.0 second. The one other state uses a PRT of 1.5 seconds to account for older drivers' reaction times.
- 13 of 20 states following the ITE equation use the 85th percentile speed (when available) as the approach speed; otherwise, the policies recommend using the posted speed limit. This applies for the timing of both the yellow change and red clearance intervals.
- 19 of 20 states following the ITE equation use a deceleration rate of 10 ft/s^2. The one other state uses a deceleration rate of 11.2 ft/s^2. This is the same state that uses a PRT of 1.5 seconds. Thus, the faster deceleration rate offsets the different PRT value.
- 20 of 20 states following the ITE equation use a vehicle length of 20 feet.
- Intersection width measurement procedures varied across all survey states.
- 9 of 22 states have left-turn treatment guidance.

ITE Survey in 2009

For the purpose of this research, and in concert with a similar project being conducted simultaneously by ITE, a survey questionnaire was developed and distributed to a sample of national and international agencies. The intent of the survey was to identify commonalities and differences in methods and factors used in yellow change and red clearance timing practices. This survey, which was distributed in June of 2009, was issued directly to the following:

- Public agency members of the Traffic Engineering, Management and Operations/Intelligent Transportation Systems (ITS), and Public Agency Councils of ITE;
- AASHTO Subcommittee on Traffic Engineering (state traffic engineers);

- A list of international organizations developed by the research team; and
- A list of agency traffic engineers generated by ITE through the National Transportation Operations Coalition.

Ultimately, the questionnaire was disseminated to approximately 2,000 recipients. A copy of the questionnaire form is included in Appendix D.

A total of 268 responses were received, 247 (92 percent) of which were from the United States and 20 (8 percent) of which were from Canada. One response was received from outside North America (Germany). Within the United States, responses were received from all 50 states except for West Virginia. Also, no response was received from the District of Columbia. Some general highlights and observations from the survey include the following:

- There is a lack of uniformity across the nation in determining the duration of yellow change and red clearance intervals. In addition to varying procedures, engineering judgment plays a significant role.
- Surprisingly, a majority of North American respondents (161 of 267, or 60 percent) indicated their agency did not have a formal policy for timing traffic signal change intervals. Sixty-two (62) percent did not have a formal policy regarding use of the red clearance interval. This lack of uniformity within agencies could be potentially problematic in terms of inconsistent signal timing for road users, and tort liability for public agencies.
- There generally is more consistency across agencies with regard to minimum values for yellow change and red clearance intervals than for maximum values. However, there still is a significant lack of consistency for both minimum and maximum values.
- More than one-half (55 percent) of respondents use posted speed limits as a factor in the calculation of change interval duration compared with 25 percent that use 85th percentile approach speeds. Agencies that do measure speeds generally update the data infrequently.
- A wide variety of procedures are used for special situations (e.g., left- or right-turn signals) or for special populations (e.g., large trucks, bicyclists).

Table 3 summarizes the methods generally used to determine the duration of change intervals. The table includes responses from agencies with and without a formal policy.

Table 4 summarizes the minimum and maximum timing values for the yellow change interval, red clearance interval, and total change interval. Key findings are as follows:

- For the yellow change interval:
  - Minimum yellow timing values ranged from 1.5 to 4 seconds, with 72 percent of respondents reporting minimum yellow timing values of 3 seconds.
  - Maximum numeric yellow timing values ranged more broadly, from 3 to 7 seconds. The largest single response (38 percent) was 5 seconds, with 77 percent of respondents reporting agency maximum yellow timing values equal to or greater than 5 seconds.
  - Seven agencies reported no maximum yellow time value.
- For the red clearance interval:
  - Minimum red timing values ranged from 0 to 2.5 seconds. The largest single response (56 percent) was 1 second, with two-thirds of respondents reporting minimum red time values equal to or greater than 1 second.
  - A broad range of maximum red timing values was reported, ranging from 1 to 6 seconds. The largest single response (50 percent) was 2 seconds, with almost two-thirds of respondents reporting agency maximum red timing values equal to or less than 2 seconds. One response (not included in Table 2) reported a maximum of 8 seconds, but limited this to a unique type of intersection, known as a single-point urban interchange, which typically has a longer intersection width.
- For the total change interval:
  - Minimum values for total change interval timing ranged from 3 to 7 seconds, with 10 agencies reporting no maximum value. The largest single response (40 percent) was 4 seconds, with 87 percent of respondents reporting minimum total change interval timing from 3 and 5 seconds.
  - Maximum numeric values for total change interval timing ranged from 5 to more than 8 seconds.

Table 3. Change interval timing methods in the absence of formal policies.

Method	Number of Responses	Percent of Responses
Kinematic Equation	85	39
Uniform Value for All Intersections	12	6
Uniform Value for All Intersections, Excluding Where Conditions Warrant an Exception	42	19
Table of Values by Approach Speed Applied to All Intersections	38	18
Other	40	18
TOTAL	217	100

Table 4. Minimum and maximum change interval timing values (number of responses).

Seconds	Yellow Interval		Red Interval		Total Change Interval	
	Minimum	Maximum	Minimum	Maximum	Minimum	Maximum
0.0	-	-	34	-	-	-
0.1 to 0.9	-	-	40	-	-	-
1.0	-	-	123	15	-	-
1.1 to 1.9	1	-	12	5	-	-
2.0	2	-	9	75	-	-
2.1 to 2.9	1	-	1	9	-	-
3.0	163	1	-	10	19	-
3.1 to 3.9	31	2	-	5	18	-
4.0	29	23	-	10	65	-
4.1 to 4.9	-	12	-	2	15	-
5.0	-	75	-	5	25	3
5.1 to 5.9	-	18	-	-	2	2
6.0	-	59	-	13	3	19
6.1 to 6.9	-	-	-	-	-	-
7.0	-	2	-	-	7	33
7.1 to 7.9	-	-	-	-	-	11
8.0	-	-	-	-	-	21
>8.0	-	-	-	-	-	16
None	-	7	2	1	10	25

– 25 agencies reported no maximum total change interval timing value.
– Agencies with maximum values for total change interval timing of 7 seconds or more, including those with no maximum, accounted for 76 percent of respondents.

Table 5 summarizes how those agencies who use the ITE kinematic equation allocate time between the yellow and red intervals.

Table 6 summarizes the values of PRT, deceleration rate, approach speed, and vehicle length used by those agencies that use the ITE kinematic equation (or variation thereof). The highlights are:

- For PRT, the largest single response (81 percent) was 1 second, with 93 percent of respondents using values for PRT of less than 2 seconds.
- More than one-half (55 percent) of the respondents use posted speed limits, compared with 25 percent that use 85th percentile approach speeds.
- For deceleration rate, the largest single response (79 percent) was 10 ft/s^2. Eleven agencies reported using a more aggressive deceleration rate of 11.2 ft/s^2.
- For vehicle length, the largest single response (69 percent) was 20 feet plus the metric equivalent of 6 meters. Eleven respondents indicated vehicle length was not used in the kinematic equation, while five said a length of zero was used – it is not clear if the latter group meant to indicate vehicle length was not used in the kinematic equation.

Table 7 summarizes the frequency of which speed measurements are updated for interval timing purposes. Approximately one-half (52 percent) of the respondents measure speeds as conditions change compared to 10 percent that measure speed only once to initially time the interval. "Other" frequencies included time periods beyond annually or as dictated by other work (e.g., speed or safety studies).

Table 8 summarizes the field measurements (other than speed) that are collected by agencies prior to timing

Table 5. Allocation of time between the yellow and red intervals when using the ITE equation.

Method	Number of Responses	Percent of Responses
The calculated value from the first two terms of the equation is allocated to the yellow interval, and the 3rd term is allocated to the red interval.	87	60
The yellow interval is set at a uniform duration and the remainder is allocated to the red interval.	11	8
The red interval is set at a uniform duration and the remainder is allocated to the yellow interval.	15	10
The entire time is allocated to the yellow interval. The red interval is not used.	0	0
Other	31	22
TOTAL	144	100

Table 6. Kinematic equation parameter values.

Parameter	Value	Number of Responses	Percent of Responses
PRT (t)	1.0 s	81	81
	1.5 s	8	8
	1.8 s	4	4
	2.0 s	2	2
	2.5 s	4	4
	3.0 s	1	1
Approach Speed (V)	Posted speed limit	133	55
	85th percentile approach speed	59	25
	Design speed	6	3
	Other	42	17
Deceleration Rate (a)	10 ft/s^2	84	79
	11.2 ft/s^2	11	10
	20 ft/s^2	1	1
	Other	11	10
Vehicle Length (L)	0	5	5
	18 ft	1	1
	20 ft	66	62
	22 ft	2	2
	25 ft	10	9
	45 ft	1	1
	Other	11	10
	Not used	11	10

Table 7. Frequency of speed measurements.

Method	Number of Responses	Percent of Responses
As conditions change	68	51
Only once, to time the interval	14	11
Annually	3	2
Other	48	36
TOTAL	133	100

Table 8. Field measurements used in change interval timing procedures.

Field Measurement	Number of Responses
Intersection width	132
Grade	30
Pedestrian volumes	27
None	26
Pedestrian crossing distance / crosswalk width	18
Traffic volumes / turning movements	9
Crash data	7
Posted speed limit	5
Measurements (non-specific) from plans or aerial photos	4
Percent trucks / heavy vehicles	4
Sight distance	4
Pedestrian characteristics	3
Bicycle volumes	1
Detector setback	1
Conflict zone for each movement	1
Intersection complexity	1
Left-turn distance to clear the intersection	1
New pedestrian generators	1
Number of approach turn lanes	1
Observed turn execution speeds for left turns without separate phasing	1
Presence of bike lanes	1
Proximity to school	1

Table 9. Special situations or populations considered when developing change interval timings.

Special Situation or Population	Number of Responses
Turning movements (most left turns)	69
Adjust as needed / engineering judgment	10
Trucks / heavy vehicles	5
Bicycles	4
Pedestrians	3
Near schools or senior centers	2
Complex / skewed intersections	2
Red-light camera enforcement	1

change intervals. Intersection width and grade (variables contained in the ITE kinematic equation) were most commonly cited. Responses also indicated that substantial consideration is given to pedestrian volumes and crossing distance.

Table 9 summarizes the special situations or populations considered by agencies when developing change interval timings. The table does not reflect the 82 respondents who said they do not consider such situations or the six respondents who said they do consider such situations but provided no details.

CHAPTER 5

Field Study Data Collection and Analysis

Objective

Prior to development of guidelines for timing the yellow change and red clearance intervals, it was first necessary to understand the behavioral characteristics of drivers approaching an intersection during the yellow and red intervals. Although relevant behavioral characteristics have been evaluated in past research, there are limitations to the current body of knowledge. Most of the fundamental driver behavior studies related to change and clearance intervals were performed more than 20 years ago. Furthermore, previous studies tended to be limited in scope due to exclusion of various driver, traffic, intersection, and/or other site-related conditions. Thus, it was determined that a thorough field investigation of driver behavior was needed in order to specify the parameters used for timing of change and clearance intervals. The field investigation focused on three issues surrounding the use of the kinematic equation:

- Determine brake-response (perception-reaction) times for use in timing of the yellow change interval,
- Determine deceleration rates for use in timing of the yellow change interval, and
- Determine approach speeds for use in timing of the yellow change and red clearance intervals for both through and left-turning vehicles.

Methodology

A naturalistic field observational study was performed to satisfy the aforementioned research objectives. In keeping with the three issues above, the primary driver behavioral characteristics that were of interest included the following:

- Brake-response (perception-reaction) times of stopping vehicles;
- Deceleration rates of stopping vehicles; and,
- Vehicular approach speeds (both through and left-turn vehicles).

Independent Factors

One of the initial tasks was to develop a comprehensive list of factors related to the field data collection that may potentially affect driver behavior during change and clearance intervals. These factors (and associated categories) were developed and prioritized based on expert opinion along with the results of the literature review, particularly the criteria suggested by Bonneson et al. (*51*). The factors were then further categorized into two groups: (1) those related to characteristics of the site and (2) those related to sampling of vehicles. The factors and categories that were used in the driver behavior evaluation are listed below.

Site-Selection Factors

- Region:
  - Southeast Michigan (metropolitan Detroit and Ann Arbor).
  - Central Florida (Orlando and The Villages area).
  - Southern California (Los Angeles and Orange County).
  - Metropolitan Washington D.C. (Northern Virginia and Maryland).
- Speed Limit:
  - ≤40 mph (low speed).
  - 45 mph.
  - ≥50 mph (high speed).
- Area Type:
  - Urban (downtown).
  - Suburban.
  - Rural (outside of incorporated area).
- Intersection Clearing Width (from stop line to far curb):
  - ≤48 feet.
  - 48 to 72 feet.

  - 72 to 96 feet.
  - 96 to 120 feet.
  - >120 feet.
- Proximity to Upstream Signal:
  - No upstream signal within 0.5 mi.
  - Upstream signal within 0.5 mi.
- Cycle Length:
  - <90 s.
  - 90 to 120 s.
  - 120 to 180 s.
  - >180 s.
- Yellow Interval Duration:
  - ≤4.0 s.
  - 4.1 to 4.5 s.
  - 4.6 to 5.0 s.
  - ≥5.1 s.
- Red Interval Duration:
  - None.
  - <1.0 s.
  - 1.1 to 2.0 s.
  - 2.1 to 3.0 s.
  - >3.0 s.
- Opposing Left-Turn Signalization:
  - Protected only.
  - Permissive only.
  - Protected permissive (leading left-turn).
  - Permissive protected (lagging left-turn).
  - None/prohibited.
- Approach Grade:
  - Level (between −3 percent and +3 percent).
  - Upgrade (greater than +3 percent).
  - Downgrade (greater than −3 percent).
- Existence of Red-Light Camera Enforcement:
  - Camera enforcement at the intersection.
  - No camera enforcement program within jurisdiction.
- Sampling Factors:
  - Time of Day/Day of Week.
  - Weekday peak periods (7–9 AM, 4–6 PM).
  - Weekday lunch period (11 AM –1 PM).
  - Weekday off-peak (all other weekday times).
  - Weekend.
- Vehicle Type:
  - Passenger vehicle (car, SUV, pickup, van, minivan).
  - Motorcycle.
  - Bus.
  - Recreational vehicle.
  - Single-unit truck.
  - Multi-unit truck.

It is important to note that demographic information of each individual driver was not obtained, as it was not possible to determine information such as the age, sex, experience, and route familiarity using the video data collection strategy. Also, it should be noted that a full-factorial study design was neither practical nor feasible, although the interactions of factors were investigated to the extent possible.

Site Selection

To provide a comprehensive set of driver behavioral data, the evaluation was scoped to include at least 80 intersection approaches evenly selected from four regions of the United States. Five states were selected for the field study to provide the necessary regional diversity, while also providing consideration for proximity to the research team. The five states used in the study included Michigan, Florida, California, Virginia, and Maryland. Virginia and Maryland were considered as a single region during site selection due to their close proximity to each other. The selection of the specific study sites was based on the following criteria:

- Adequate representation within each of the site-related factors,
- Adequacy of the site for camera placement,
- Approaches that were relatively straight,
- Two through lanes on the approach separated by visible lane-line markings,
- Intersections with approximately 90-degree angle approaches, and
- Agency cooperation and/or assistance.

Potential study sites were initially identified by the research team based on familiarity with the locations or by assistance from local transportation jurisdiction officials (i.e., state, county, or city). The team also relied on available internet imagery (e.g., Google Earth) to assist in site selection. No more than two approaches were utilized at a single intersection, and in some cases only a single approach was used. It was desirable to select adjacent approaches at each intersection to include both the major and minor roadways. However, conditions did not always allow for this to occur and thus, two opposing approaches on the same roadway were often utilized.

Eighty-three sites distributed between the four regions were ultimately utilized in the field evaluation. Note that for this study, the term "site" refers to a single intersection approach. The primary characteristics of the study sites are summarized in Table 10. Detailed site characteristics within each state are presented in Appendix E.

Although it was not possible to obtain specific driver information during the field study, accommodation of certain driver-related issues such as age and familiarity was inferred through the site-selection process. For example, several sites were selected near a major Florida retirement community—The Villages—with the intent of having a larger sample of elderly drivers. Similarly, several sites were selected near

Table 10. Summarized categorical representation of the study sites by factor.

Factor	Category	Total Number of Sites	Percent of All Sites
State	Michigan	23	28
	Florida	20	24
	California	21	25
	Virginia	11	13
	Maryland	8	10
Speed Limit	≤40 mph (low speed)	30	36
	45 mph	36	43
	≥50 mph (high speed)	17	20
Area Type	Urban (downtown)	6	7
	Suburban	70	84
	Rural (outside of incorporated boundaries)	7	8
Clearing Width (from stop line to far curb)	≤48 ft	3	4
	48 to 72 ft	8	10
	72 to 96 ft	22	27
	96 to 120 ft	17	20
	>120 ft	33	40
Proximity to Nearby Signal	No Upstream Signal Within 0.5 mi	21	25
	Upstream Signal Within 0.5 mi	62	75
Cycle Length Range	<90 s	22	27
	90 to 120 s	13	16
	120 to 180 s	35	42
	>180 s	13	16
Yellow Interval	≤4.0 s	26	31
	4.1 to 4.5 s	29	35
	4.6 to 5.0 s	21	25
	≥5.1 s	7	8
All-Red Interval	None	9	11
	<1.0 s	29	35
	1.1 to 2.0 s	33	40
	2.1 to 3.0 s	10	12
	>3.0 s	2	2
Opposing Left-Turn Signalization	Protected only	58	70
	Permissive only	4	5
	Protected permissive	7	8
	Permissive protected	4	5
	Prohibited/None	10	12
Grade	Downgrade (greater than -3 percent)	2	2
	Level (between -3 percent and +3 percent)	78	94
	Upgrade (greater than +3 percent)	3	4
Red-Light Camera Enforcement	Camera Enforcement on the Approach	10	12
	No Camera Enforcement in Jurisdiction	73	88

major theme parks and sports stadiums in greater Orlando and southern California with the intent of having a larger sample of drivers unfamiliar with the sites.

Field Data Collection Procedures

Behavioral data were collected at the 83 signalized intersection approaches between October 2009 and July 2010. Vehicles were recorded while approaching an intersection using a high-definition video camera mounted on a modular 20-foot tall aluminum pole that was securely strapped and locked to a rigid roadside post. An example of the video camera setup is shown in Figure 3.

The camera mounting system could be installed or removed in as little as 10 minutes. Using this setup, it was possible to obtain up to 8 hours of continuous, unattended recording before depleting the battery, although for efficiency purposes, between 3 and 5 hours of video were typically obtained per site before moving the camera to another location. During a given data collection event, all necessary data were recorded at the approach using a single video camera. A maximum of four video cameras was utilized during the data collection activities, allowing for up to four intersection approaches to be recorded at any given time.

After securing and fully extending the pole, the camera's field of view was verified using a portable television monitor

Figure 3. Field setup for video recording of driver behavior data.

that was connected to the camera using a long RCA patch cable. Minor adjustments to the field of view, such as turning the pole/camera left or right, could be made while observing the viewing screen of the monitor. However, coarse adjustments to the field of view, such as upward or downward tilt or telephoto zoom, required the camera be brought back to the ground for repositioning or other manipulation.

The camera was installed upstream of the intersection on each subject approach to provide a field of view ranging from between 300 feet and 600 feet along the intersection approach, depending on speed limit. High-speed approaches required the greatest viewing distance (i.e., 600 feet) as the dilemma zone occurs farther upstream, whereas low-speed approaches required a much shorter field of view (i.e., 300 feet). The cameras were affixed to an adequately located roadside post and were aimed downstream toward the intersection so that the necessary approach distance, intersection, and traffic signals were in full view. From this vantage point, the cameras allowed for observation of all characteristics relevant to the behavioral evaluation, including the traffic signal indications, brake light indications, location of the vehicle with respect to the stop line, vehicle type, spacing between successive vehicles, whether the vehicle stopped or went through, and whether red-light running occurred. A general schematic of a typical video camera installation is shown in Figure 4.

To provide for efficiency in data collection and to reduce the potential for "double counting" vehicles, with few exceptions, all data collection for a particular approach was performed within a single day. Data collection was typically only performed during dry conditions and during daylight hours. Only a limited number of sites included data collected during wet weather and/or during periods shortly after dusk. This was because of the potential damage to the video camera during periods of heavy rain along with the inability to record high quality video at night. However, past studies have shown no significant difference in either PRT or deceleration rate between dry and wet weather conditions (*52, 53*). Additionally, these studies have shown no significant effect on the probability of stopping versus going based on weather condition.

Data Extraction

The videos were immediately transferred to a computer after each data collection event. As the videos were digitally recorded to a solid state flash memory card, no additional conversion was needed to be viewed on a computer. The videos were manually reviewed using QuickTime software to extract the relevant behavioral data during each change and clearance interval for the subject approach. QuickTime allowed for frame-by-frame review of the videos to determine the relevant vehicular location and time information. The video was recorded at a rate of 60 frames per second, allowing time to be recorded to the nearest 0.0167 seconds, as displayed in the video player. The regular pattern of the white broken lane-line pavement markings (e.g., 10-foot marking with 30-foot gap) provided convenient field reference markers for determining the location of a vehicle with respect to the stop line, which was used as the primary reference point at each site. The nominal lane-line striping intervals varied by state and ranged between 24 feet and 50 feet as follows:

- 24 feet (California);
- 40 feet (Florida, Virginia, Maryland, Michigan [urban]); and
- 50 feet (Michigan [rural and suburban]).

Field measurements of the actual length of the lane-line pavement markings and the gap between them allowed for a grid to be overlaid onto the computer screen, which provided a scale by which vehicle positioning with respect to the stop line could be determined to approximately the nearest five feet. Field verification of the distance between successive lane-line markings showed relatively consistent placement, typically within ±0.5 feet of the nominally specified distance. Figure 5 displays a screenshot of the field of view provided from an example intersection video frame.

During the video review process, data were obtained during each signal cycle for each non-turning vehicle that was either the last vehicle to go through the intersection or the first vehicle to stop in each lane on the subject approach. An example depicting the designation of last-to-go and first-to-stop vehicles is shown in Figure 6. Turning vehicles were excluded as drivers must typically decelerate to complete the turning maneuver. Thus, brake-response times and deceleration rates for turning vehicles are not necessarily indicative of a response to the yellow indication. Note, however, that a sample of left-turning vehicle speeds was collected for the assessment of left-turn approach speeds.

The following information was recorded from the video for each subject vehicle included in the sample:

- Amount of time to traverse the initial speed measurement zone;
- Time and location at the start of yellow;
- Action of the vehicle:
  - Stopped,
  - Went through, reached the stop line prior to the end of the yellow,
  - Went through, reached the stop line after the end of the yellow (i.e., red-light running),
- Time and location at the start of brake light illumination (stopping vehicles only);

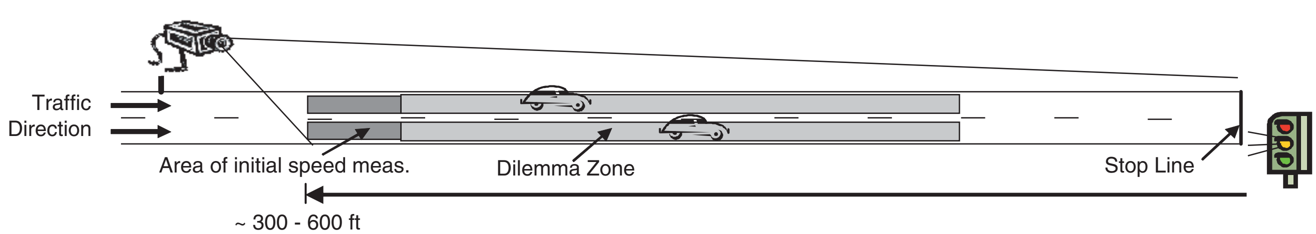

Figure 4. Video recording field of view.

Figure 5. Example screenshot from intersection video overlaid with simulated grid.

- Time that the vehicle stopped (stopping vehicles only);
- Intersection entry time after the start of red (red-light running vehicles only);
- Whether the vehicle was a platoon leader, platoon follower, or non-platooned based on the following spacing requirements:
  - 100 ft at locations with speed limits of 40 mph and below,
  - 150 ft at locations with speed limits of 45 mph and above,
- Presence of an opposing left-turning vehicle;
- Time of day; and
- Vehicle type.

Vehicles were excluded for any of the following reasons:

- Was not a first-to-stop or last-to-go vehicle within the respective lane,
- Turned right or left or u-turn at the intersection,
- Began braking prior to the onset of yellow (stopping vehicles only),
- Turned out of a driveway within the camera's view,

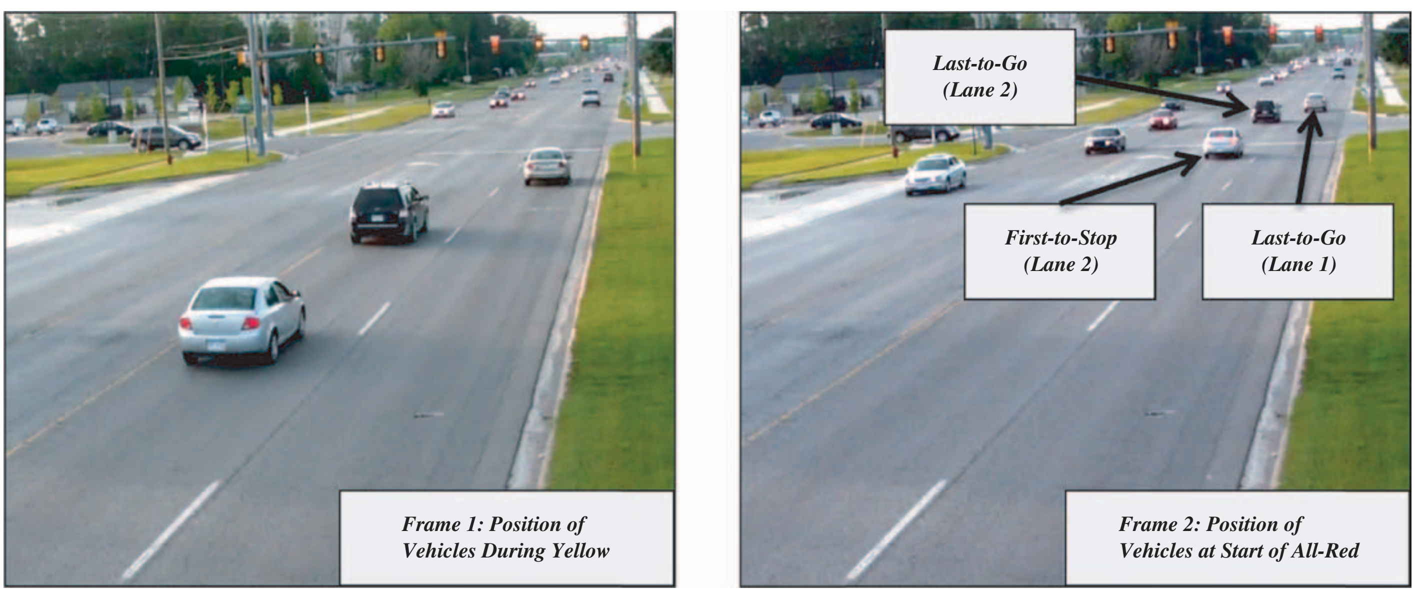

Figure 6. Designation of last-to-go and first-to-stop vehicles by lane.

- Approached at an unusually low rate of speed (<15 mph),
- Unusual or erratic behavior, or
- Presence of a queue on the subject approach.

Red-light running events were defined as cases where the front of the vehicle reached the stop line after the onset of the red indication. This definition follows the "permissive" yellow rule and is consistent with the *Uniform Vehicle Code* (*16*).

All time-related information was recorded in terms of the actual video frame, which was easily converted to time based on the recorded frame-rate of 60 frames per second. Brake light indications and traffic signal indications were defined based on the first video frame that illumination of any of the respective displays could be visibly detected. Vehicle location estimates were made with respect to the front wheels. During the stop-time assessment for a braking vehicle, the technicians were trained to pause the video immediately prior to full cessation of motion to account for the slight perception lag that would occur upon identification of the vehicles true stop time. This time was recorded onto the data collection form and the video was immediately played to determine if the vehicle had truly stopped or if any additional forward motion occurred, in which case, the measurement was discarded. Determination of the time that the vehicle stopped was made based on convergence of three independent trials by the video review technician that were each within 10 frames (1/6th of one second) of each other. The average of the three trials was recorded as the vehicle's stop time. Occasionally, certain vehicular data, such as brake light indications, positioning information, or the time that the vehicle stopped, could not be obtained for a given subject vehicle, which resulted in an incomplete record for the particular vehicle. Such occurrences were caused by a variety of factors, but most commonly occurred due to obstructions from other approaching vehicles or sun glare.

Data Reduction

More than 328 hours of video were obtained—an average of nearly 4 hours per each of the 83 sites. The raw time and positioning information obtained from the videos was used to compute approach speeds and to estimate travel time to the intersection, brake-response (perception-reaction) times, and deceleration rates. Approach speeds prior to the start of yellow were computed using the following equation:

$$S = \frac{d_S}{t_S} \qquad \textbf{Equation 6}$$

Where:

S = approach speed prior to start of yellow (ft/s),
d_S = speed measurement distance (ft), and
t_S = time to traverse the speed measurement distance (s).

The travel time to the stop line at the onset of yellow was estimated by dividing the subject vehicle's distance from the stop line at the onset of yellow by its approach speed. Note that this provided only a prediction of the travel time based on the approach speed and distance upstream and was not the *actual* travel time, which may vary due to acceleration or deceleration. The following equation was used to calculate the predicted travel time to the stop line at the start of yellow:

$$TT = \frac{d_Y}{S} \qquad \textbf{Equation 7}$$

Where:

TT = predicted travel time to stop line at start of yellow (s),
d_Y = distance from the stop line at start of yellow (ft), and
S = approach speed prior to start of yellow (ft/s).

Brake-response times were computed as the absolute difference between the time at start of yellow and the time when the brake lights became visible, using the following equation:

$$BRT = t_{BL} - t_Y \qquad \textbf{Equation 8}$$

Where:

BRT = brake-response time (s),
t_{BL} = time when brake lights became visible (s), and
t_Y = time at start of yellow (s).

Brake-response time is a common field-measured estimate of PRT. Although the two values are similar, brake-response time is typically considered to be slightly longer than PRT as it includes any lag time between the driver removing his/her foot from the accelerator and applying the brake (i.e., "coasting"), which could not be quantified from the videos. It was also not possible to determine cases in which the driver prematurely removed his/her foot prior to the start of yellow, but did not apply the brakes until after the yellow.

The *average* deceleration rate was computed for each vehicle based on the approach speed and braking time. Although it is acknowledged that drivers may use variable deceleration rates during braking, it was not practical to measure changes in deceleration rate for individual subject vehicles. Because the speed measurements were taken immediately prior to the onset of yellow, any impacts on speeds caused by coasting prior to braking were considered to be negligible. Braking time was computed as the absolute difference between the time that the brake lights became visible and the time that vehicle had stopped. The following formula was used to compute the average deceleration rate:

$$D_{avg} = \frac{S}{t_{stop} - t_{BL}} \qquad \textbf{Equation 9}$$

Where:

D_{avg} = average deceleration rate (ft/s^2),

S = approach speed prior to start of yellow (ft/s),

t_{stop} = time that the vehicle had stopped (s), and

t_{BL} = time when brake lights became visible (s).

Establishment of Dilemma Zone Boundaries

The vehicular observation data were tabulated, organized, and coded into a single data file for detailed analyses. The additional independent variables were appropriately coded for each of the subject vehicles. The full data set included 7,482 vehicle records collected from the 83 sites. The data set was then further stratified to include a more concise range of vehicular arrivals to more accurately represent "dilemma zone" driver behavior. The dilemma zone (or more accurately termed the "decision zone" for this study) has historically been defined in the literature as the area upstream of the intersection between which 10 percent and 90 percent of drivers (*5*) will stop in response to the yellow indication. Previous research has shown that the 10 percent to 90 percent stop region typically corresponds to drivers that are between 2.5 seconds and 5.5 seconds upstream of the intersection at the start of yellow (*6*).

Initial investigation of the full data set showed good agreement with the 2.5 second to 5.5 second dilemma zone region, as 7.9 percent of drivers stopped when approximately 2.5 seconds upstream at the start of yellow and 93.1 percent of drivers stopped when approximately 5.5 seconds upstream. The distribution of driver actions versus travel time to the stop line at the start of yellow is displayed in Table 11 and graphically in Figure 7.

To provide further confirmation of the dilemma (decision) zone boundaries, a basic binary logistic regression model was developed from the full data set to predict the probability of a driver stopping as a function of travel time to the intersection at the start of yellow. Two forms of the logistic regression model were formed: one based solely on travel time and another including both travel time and speed. The results from the travel time–dependent model are shown in Figure 8.

As expected, travel time was found to be a highly significant variable for predicting driver action. The travel times related to stopping probabilities of 10 percent and 90 percent were approximately 2.5 seconds and 5.2 seconds, providing good agreement with previous research. The speed-dependent model showed little variation in the 10 percent or 90 percent travel time boundaries with respect to approach speed. The travel times related to a 10 percent stopping probability were 2.4 seconds and 2.6 seconds for 25 mph and 55 mph speeds, respectively. Similarly, the travel times related to a 90 percent stopping probability were 5.1 seconds and 5.3 seconds for 25 mph and 55 mph speeds, respectively. Thus, to provide an accurate representation of dilemma zone driver behavior and to be consistent with previous research, the data set was filtered to only include the following:

- Last-to-go vehicles (including red-light runners) that were greater than 2.5 seconds upstream of the intersection at the start of yellow and
- First-to-stop vehicles that were less than 5.5 seconds upstream of the intersection at the start of the yellow.

The data set included 4,820 vehicles based on these dilemma zone boundaries, which included the following:

- 2,606 vehicles that stopped (54.1 percent),
- 2,087 vehicles that went through and entered prior to the red (43.3 percent), and
- 127 red-light running vehicles (2.6 percent).

Table 11. Distribution of driver actions versus travel time to stop line at start of yellow.

Travel Time (seconds) to Stop Line at Start of Yellow		Percent of Vehicles		Total Vehicle Count
Range	Midpoint	Stopped	Went Through	
1.75 – 2.25	2.0	4.7	95.3	407
2.25 – 2.75	**2.5**	**7.9**	**92.1**	**687**
2.75 – 3.25	3.0	17.7	82.3	755
3.25 – 3.75	3.5	37.6	62.4	795
3.75 – 4.25	4.0	56.4	43.6	864
4.25 – 4.75	4.5	73.8	26.2	833
4.75 – 5.25	5.0	86.9	13.1	766
5.25 – 5.75	**5.5**	**93.1**	**6.9**	**710**
5.75 – 6.25	6.0	96.8	3.2	505

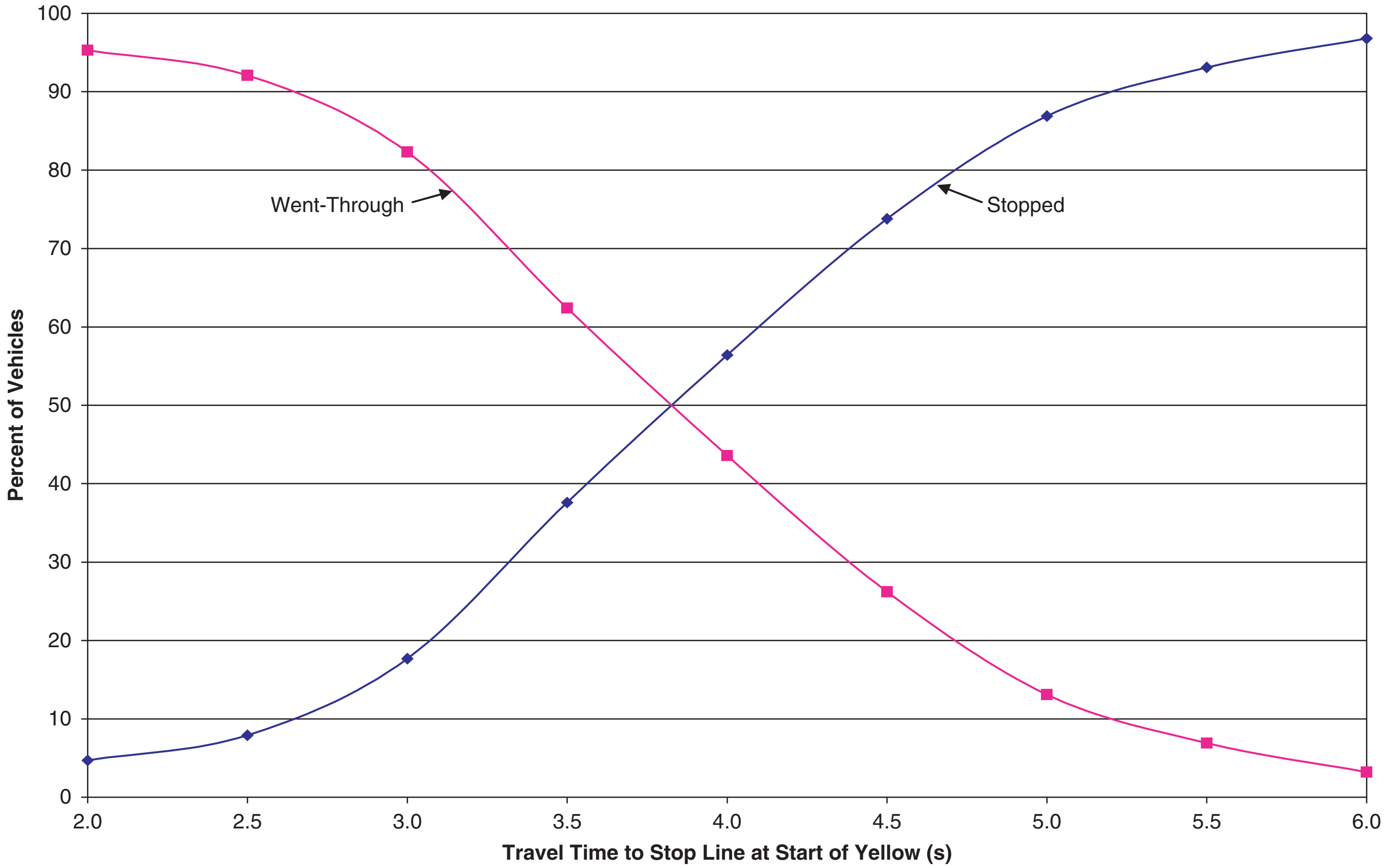

Figure 7. Distribution of driver actions versus travel time to stop line at start of yellow.

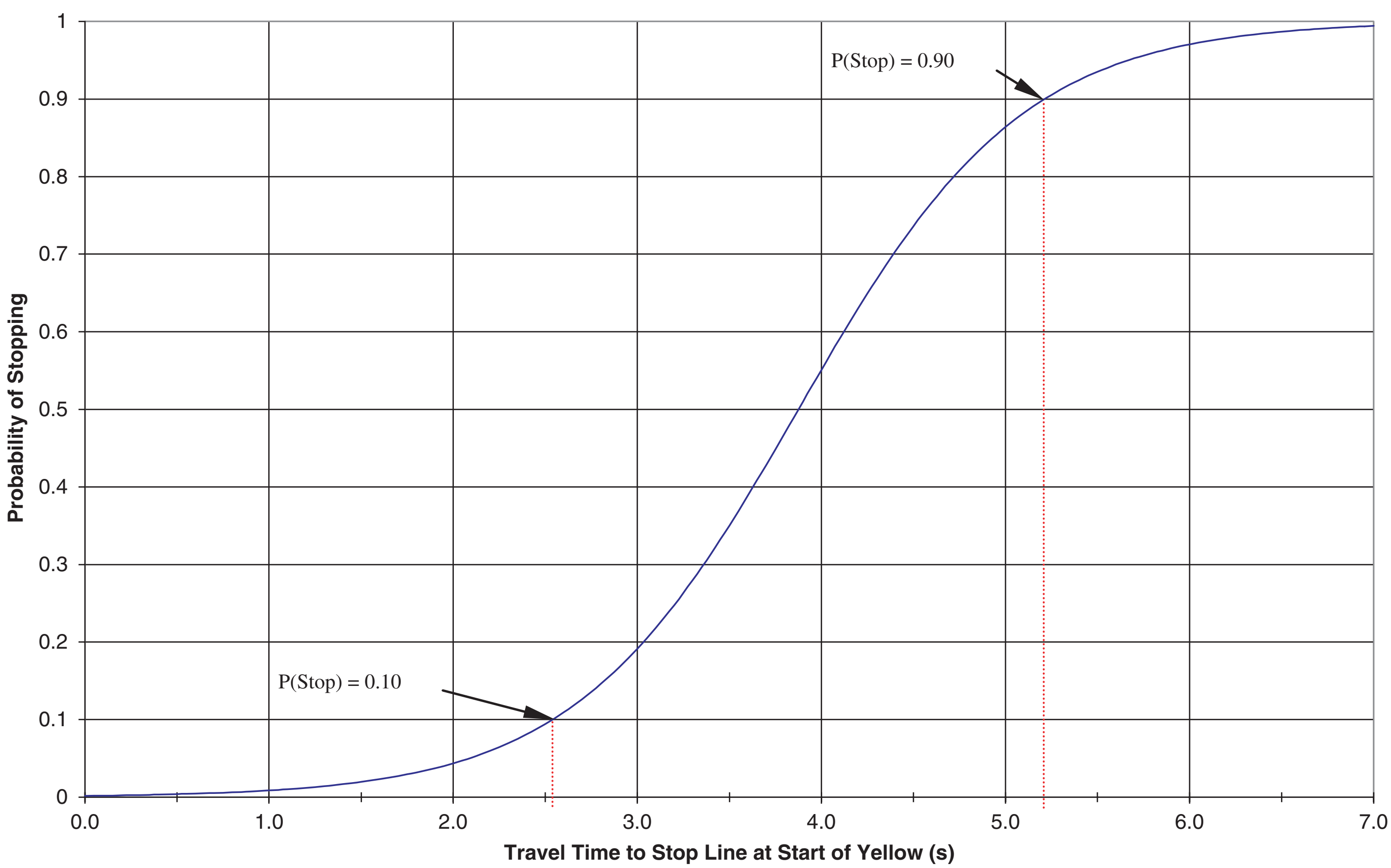

Figure 8. Probability of stopping (logistic regression) versus travel time to stop line at start of yellow.

Analyses

To determine any obvious trends, sources for potential bias, and distributions of the data, descriptive statistics and simple graphical representations were investigated before any analyses were performed. Vehicles arriving during wet pavement conditions were excluded from the primary data set. The analyses were performed using a multi-factor analysis of variance (ANOVA) in SPSS version 18 (*54*). The independent variables included both those related to the characteristics of the location and the sampling variables. The dependent variables for these analyses included the following:

- Brake-response time for first-to-stop vehicles,
- Deceleration rate for first-to-stop vehicles, and
- Approach speeds for both through vehicles and left-turn vehicles.

Results: Brake-Response Time and Deceleration Rate

Descriptive Statistics

Figures 9 and 10 display the cumulative distributions of brake-response times and deceleration rates, respectively, for the first-to-stop vehicles observed in this study. Also included in the figures, for comparative purposes, are the distributions of data reported in select previous studies. The basic descriptive statistics for brake-response time and deceleration rate are shown in Table 12.

As observed in Figure 9, the distributions of brake-response times observed in this study were in good agreement, albeit slightly shorter, compared to data observed in previous studies (*23, 38, 52, 55*). The mean brake-response time of 1.00 seconds observed here was identical to the default PRT value of 1.0 seconds recommended by ITE for timing of the yellow interval (*56, 57*).

The deceleration rates observed in this study were equally, if not more, similar to values observed in previous research. Figure 10 shows a nearly identical distribution of deceleration rates for the data observed in this study compared to a 2007 Wisconsin study by Gates et al. (*23*). The 50th percentile deceleration rate observed here was very similar to the 50th percentile values found in each of the previous studies, with the exception of the 1983 study by Wortman and Matthias (*55*). The overall mean deceleration rate of 10.08 ft/s^2 is very close to ITE's recommended deceleration rate of 10 ft/s^2 (*56, 57*).

Significant Factors

The ANOVA analysis found several variables that significantly affected brake-response time and/or deceleration

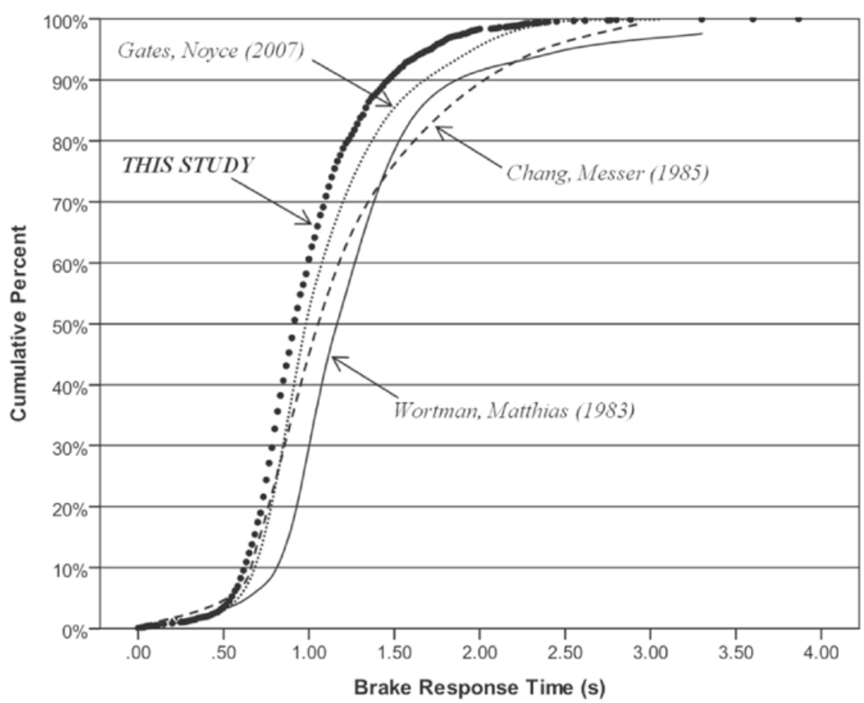

Figure 9. Brake-response times of first-to-stop vehicles.

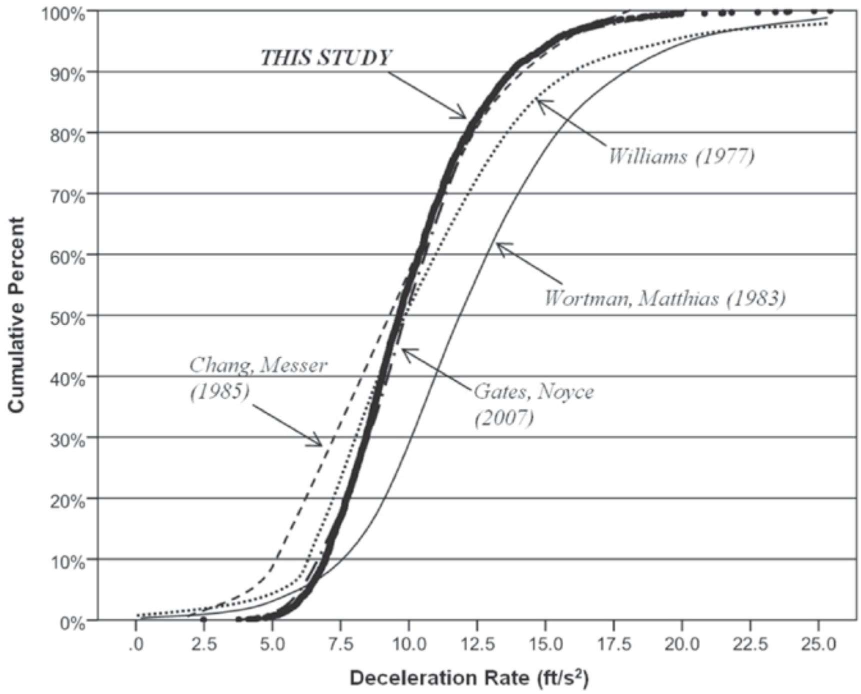

Figure 10. Deceleration rates of first-to-stop vehicles.

rate. The variables were classified as highly significant, significant, or insignificant based on the relative magnitude of the F-statistic provided in the ANOVA results. The significance of each factor investigated in the ANOVA is shown in Table 13.

Table 13 shows that the most highly significant factors were those related to the vehicle's time from the intersection and the behavior of the driver. Most of the site-related characteristics or temporal characteristics were not significant. Further investigation of each significant factor revealed the following:

- Brake-response times
  - decreased as approach speed increased (i.e., faster drivers reacted more quickly);
  - increased as travel time to the intersection at the start of yellow increased (i.e., drivers reacted more slowly when farther from the intersection);
  - increased as the deceleration rate increased (i.e., drivers decelerating more rapidly used longer brake-response times); and
  - decreased for steep downgrades.
- Deceleration rates
  - increased as approach speed increased (i.e., faster drivers used greater deceleration);
  - decreased as travel time to the intersection at the start of yellow increased (i.e., drivers used lower deceleration when farther from the intersection);
  - increased as the brake-response time increased (i.e., slower-reacting drivers used greater deceleration rates);
  - increased as speed limit increased (i.e., sites with higher speed limits had greater deceleration);
  - increased as yellow duration increased (i.e., sites with longer yellow intervals had greater deceleration); and
  - increased for steep upgrades and decreased for steep downgrades.

Table 12. Basic descriptive statistics for brake-response time and deceleration rate.

		Brake-Response Time (s)	Deceleration Rate (ft/s^2)
Number of Vehicles		2,422	2,458
Mean		1.00	10.08
Standard Deviation		0.37	2.83
Percentiles	15th	0.68	7.32
	50th	0.92	9.65
	85th	1.33	12.89

The descriptive statistics for brake-response time and deceleration rate categorized by each of the significant factors are shown in Tables 14 through 18 and graphically in Figures 11 through 15. Discussion and recommendations with respect to timing of the yellow interval are provided in the section that follows.

Table 13. Statistical significance of independent factors for brake-response time and deceleration rate.

	Highly Significant Factors	Significant Factors	Insignificant Factors
Brake-Response Time	• Travel Time to Stop Line at Start of Yellow • Approach Speed • Deceleration Rate • Approach Grade*	• Speed Limit	• Vehicle Type • Peak vs. Off-Peak • Weekday vs. Weekend • Signal Proximity • Clearing Width • Presence of Opposing Left-Turner • Platoon vs. Free-Flowing • Cycle Length • Yellow Interval Duration
Deceleration Rate	• Travel Time to Stop Line at Start of Yellow • Approach Speed • Brake-Response Time • Approach Grade*	• Speed Limit • Yellow Interval Duration	• Vehicle Type • Peak vs. Off-Peak • Weekday vs. Weekend • Signal Proximity • Clearing Width • Presence of Opposing Left-Turner • Platoon vs. Free-Flowing • Cycle Length

*Limited sample size

Table 14. Brake-response times and deceleration rates by vehicular approach speed.

Range of Vehicular Approach Speeds			Brake-Response Time (s)	Deceleration Rate (ft/s^2)
20 to 30 mph	Number of Vehicles		263	263
	Mean		1.07	8.13
	Standard Deviation		0.38	2.52
	Percentiles	15	0.77	5.89
		50	1.02	7.52
		85	1.42	10.68
30 to 40 mph	Number of Vehicles		761	763
	Mean		1.03	9.26
	Standard Deviation		0.39	2.43
	Percentiles	15	0.72	7.02
		50	0.95	8.78
		85	1.38	11.36
40 to 50 mph	Number of Vehicles		999	1022
	Mean		0.97	10.46
	Standard Deviation		0.36	2.64
	Percentiles	15	0.67	7.91
		50	0.90	10.03
		85	1.30	13.06
50 to 60 mph	Number of Vehicles		344	353
	Mean		0.94	11.90
	Standard Deviation		0.35	2.67
	Percentiles	15	0.63	9.53
		50	0.88	11.43
		85	1.25	14.50

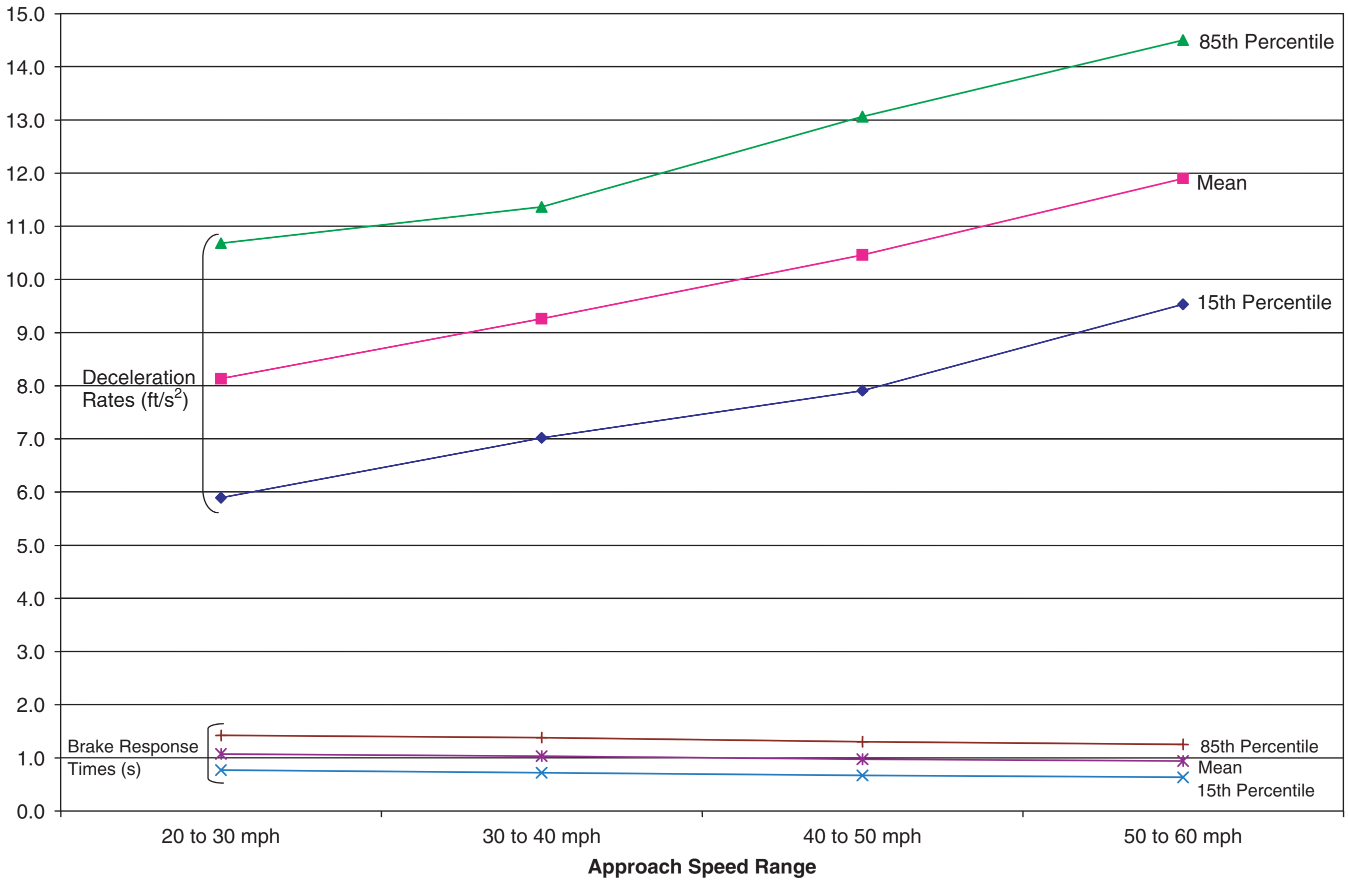

Figure 11. Brake-response times and deceleration rates by vehicular approach speed.

Table 15. Brake-response times and deceleration rates by travel time to stop line at start of yellow.

Travel Time to Stop Line at Start of Yellow			Brake-Response Time (s)	Deceleration Rate (ft/s^2)
<4.5 s	Number of Vehicles		1206	1225
	Mean		0.93	11.28
	Standard Deviation		0.33	2.94
	Percentiles	15	0.65	8.50
		50	0.87	10.93
		85	1.22	14.10
>4.5 s	Number of Vehicles		1216	1233
	Mean		1.06	8.90
	Standard Deviation		0.40	2.14
	Percentiles	15	0.73	6.85
		50	0.98	8.63
		85	1.42	10.93

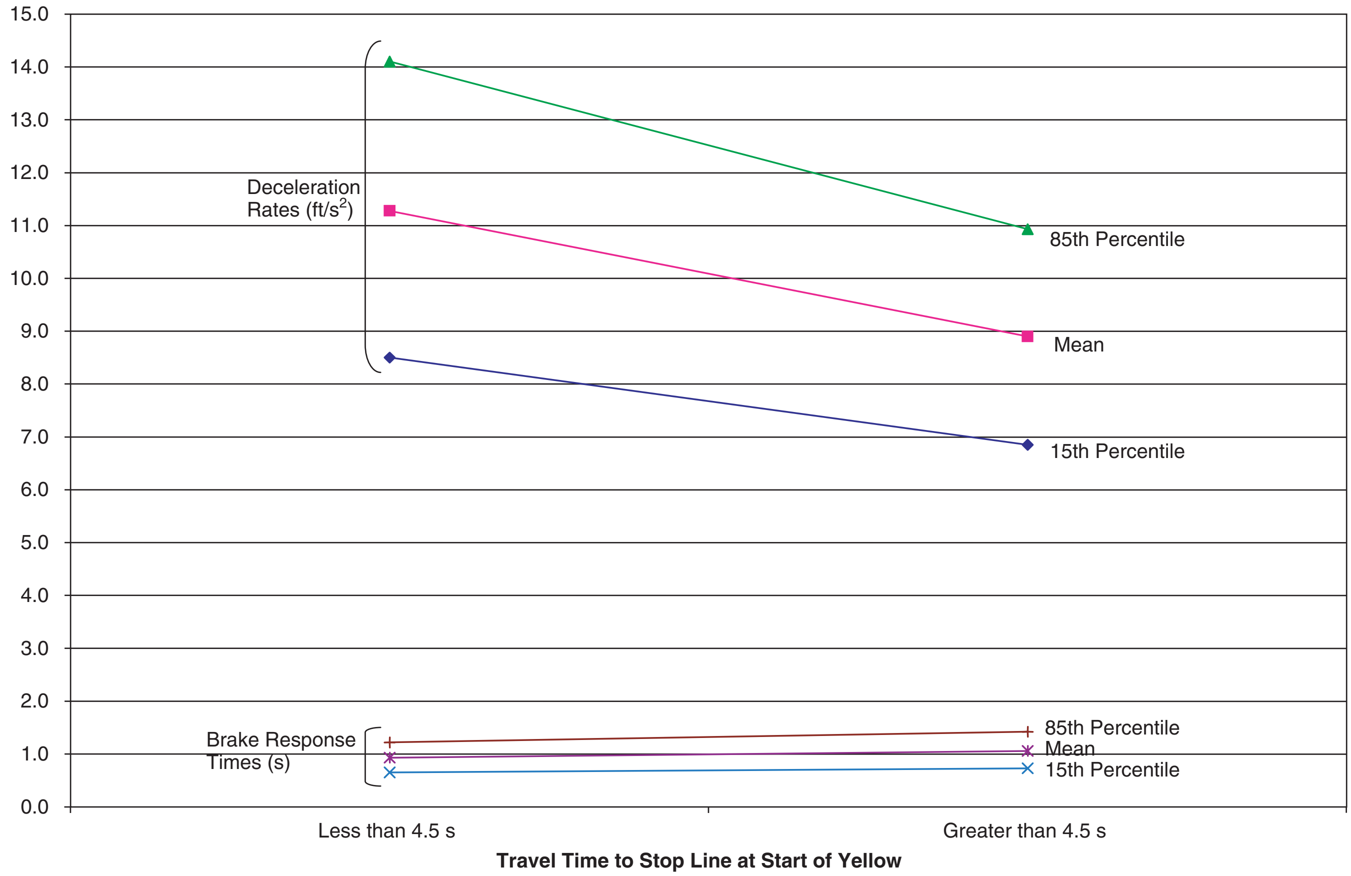

Figure 12. Brake-response times and deceleration rates by travel time to stop line at start of yellow.

Table 16. Deceleration rates by speed limit.

Speed Limit Range			Deceleration Rate (ft/s^2)
25 mph to 30 mph	Number of Vehicles		267
	Mean		8.21
	Standard Deviation		2.24
	Percentiles	15	6.17
		50	7.85
		85	10.67
35 mph to 40 mph	Number of Vehicles		540
	Mean		9.79
	Standard Deviation		2.80
	Percentiles	15	7.32
		50	9.21
		85	12.31
45 mph to 50 mph	Number of Vehicles		1444
	Mean		10.29
	Standard Deviation		2.78
	Percentiles	15	7.63
		50	9.83
		85	13.10
55 mph to 60 mph	Number of Vehicles		207
	Mean		11.80
	Standard Deviation		2.53
	Percentiles	15	9.50
		50	11.42
		85	14.18

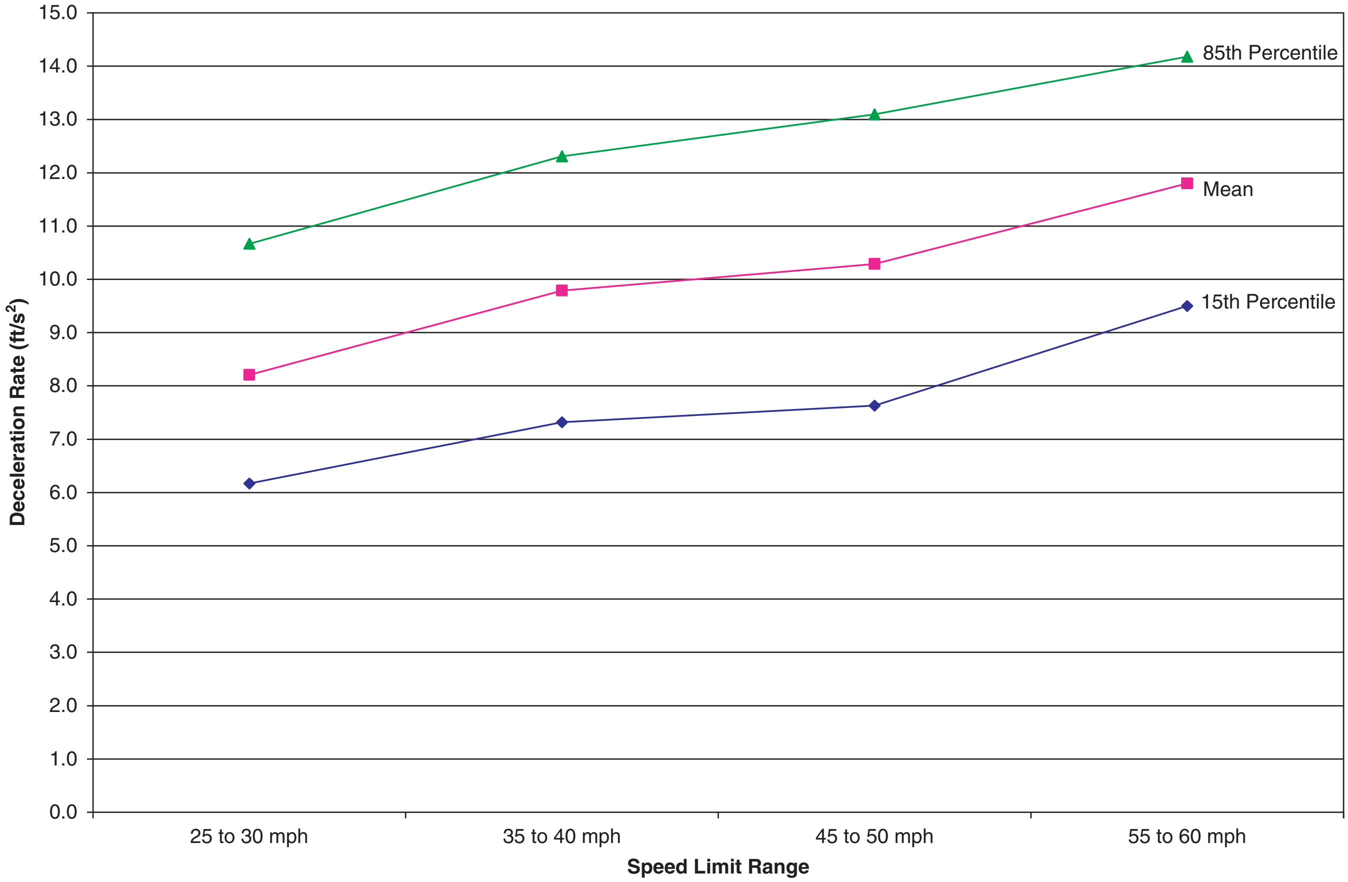

Figure 13. Deceleration rates by speed limit.

Table 17. Deceleration rates by yellow duration.

Yellow Duration			Deceleration Rate (ft/s^2)
≤ 4 s	Number of Vehicles		591
	Mean		9.21
	Standard Deviation		2.69
	Percentiles	15	6.75
		50	8.69
		85	11.70
4.1–4.9 s	Number of Vehicles		1082
	Mean		10.06
	Standard Deviation		2.76
	Percentiles	15	7.36
		50	9.66
		85	12.83
≥ 5 s	Number of Vehicles		785
	Mean		10.77
	Standard Deviation		2.86
	Percentiles	15	8.11
		50	10.32
		85	13.49

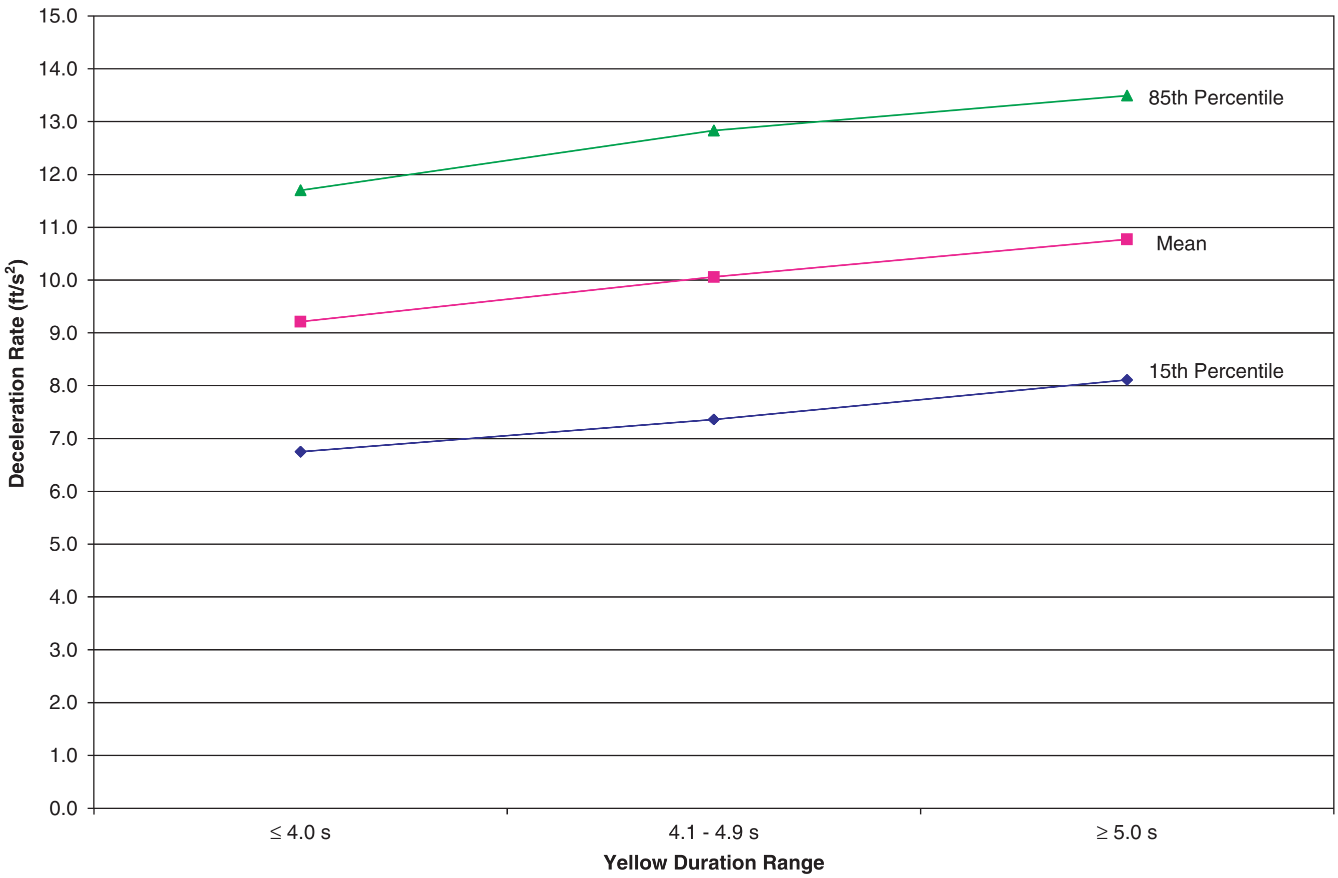

Figure 14. Deceleration rates by yellow duration.

Table 18. Brake-response times and deceleration rates by approach grade category.

Grade Category			Brake-Response Time (s)	Deceleration Rate (ft/s²)
Downgrade >-3 percent	Number of Vehicles		7	44
	Mean		0.66	9.41
	Standard Deviation		0.42	2.00
	Percentiles	15	0.21	7.43
		50	0.62	8.85
		85	1.21	12.13
Level	Number of Vehicles		93	93
	Mean		1.03	10.33
	Standard Deviation		0.42	2.66
	Percentiles	15	0.70	7.98
		50	0.92	9.83
		85	1.50	12.60
Upgrade >3 percent	Number of Vehicles		127	127
	Mean		1.05	12.02
	Standard Deviation		0.34	3.22
	Percentiles	15	0.75	8.56
		50	0.97	11.59
		85	1.40	15.47

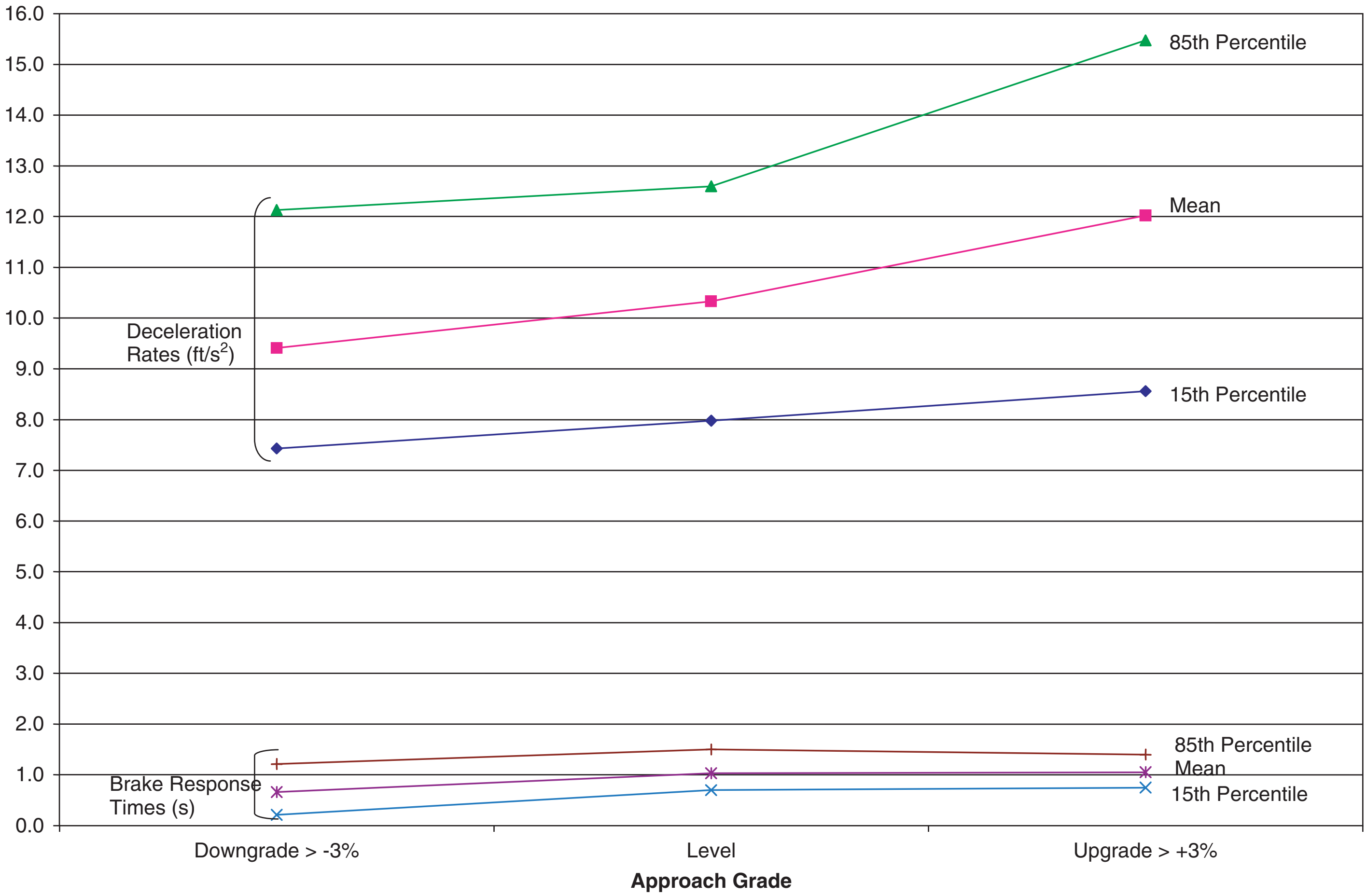

Figure 15. Brake-response times and deceleration rates by approach grade category.

Results: Approach Speed Versus Speed Limit

Questions often arise when selecting the appropriate operating speed for use in timing of the yellow change and red clearance intervals. While the 85th percentile speed is typically recommended (*57*), agencies often do not have the resources available to collect approach speed data. Additionally, there is little guidance as to appropriate methods for collecting approach speed data. In lieu of locally collected data, agencies often use the approach speed limit when calculating yellow change and red clearance intervals.

It is also important to recognize that the approach speed characteristics of left-turning vehicles are often different than for through-moving vehicles. Except for shallow turning paths, left-turning vehicles must typically decelerate to safely complete the left-turn maneuver and brake-response is not necessarily related to the yellow change interval. As such, the braking characteristics of left-turning drivers are different than for drivers of through-moving vehicles with respect to the yellow indication.

In light of these issues, an important research objective included evaluation of both through-moving vehicle speeds and left-turning vehicle speeds with respect to the posted speed limit. The primary outcome was to develop a rule-of-thumb approach speed recommendation for use in the calculation of yellow change and red clearance intervals in lieu of 85th percentile speed data.

Through-Moving Vehicles

Approach speed data were randomly sampled for a total of 3,632 free-flowing through-moving vehicles. The vehicles were randomly sampled from 60 of the study sites, which included speed limits ranging between 25 mph and 55 mph. At least four sites from each speed limit category were included and at least 15 speed samples were obtained per site. Speed data were collected for each of the five states used in the study. Speeds were measured between 300 and 600 feet upstream of the intersection using the aforementioned video data collection procedure. Both go-through and stopping vehicles were sampled, as long as the brakes were not applied either before or during the speed measurement. Wet pavement conditions and non-free-flowing

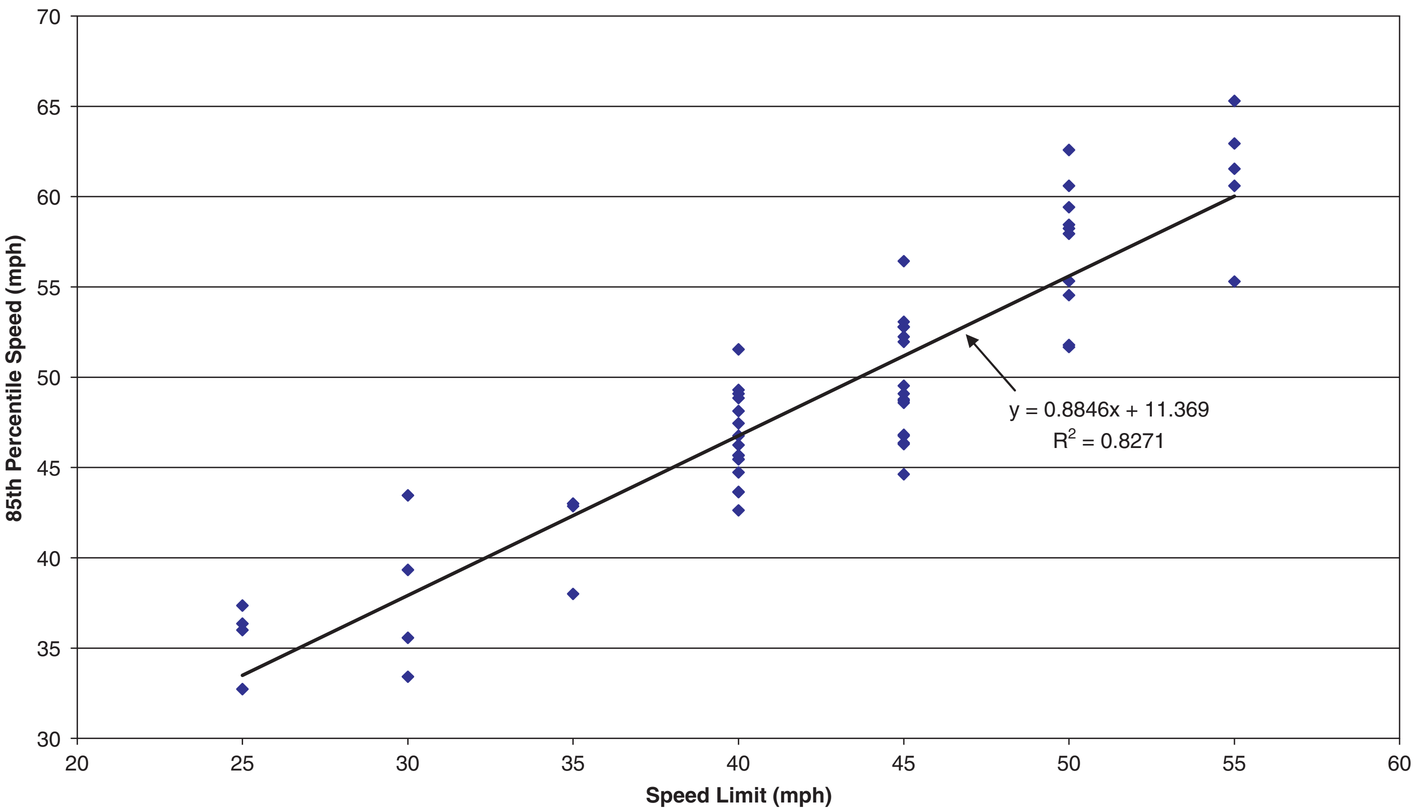

Figure 16. Simple linear regression for 85th percentile speed versus speed limit.

vehicles were excluded. The mean and 85th percentile speed statistics were calculated for each speed limit category using three different methods:

1. Average of the mean and 85th percentile speed values of each location within the respective speed limit category,
2. The mean and 85th percentile of the speeds pooled within the respective speed limit category, and
3. Least squares regression (simple linear) for mean and 85th percentile speed values of each location versus speed limit.

The simple least squares linear regression model showed very good fit for both the mean and 85th percentile speed data plotted versus speed limit for each of the 60 sites. The regression results for the 85th percentile speed are shown in Figure 16. The regression equations for prediction of the mean and 85th percentile speed, respectively, as a function of the approach speed limit, were given as follows:

$$S_{mean} = 0.8262 \times Speed\,Limit + 7.745 \qquad R^2 = 0.786 \quad \textbf{Equation 10}$$

$$S_{85th} = 0.8846 \times Speed\,Limit + 11.369 \qquad R^2 = 0.827 \quad \textbf{Equation 11}$$

Table 19 and Figure 17 display the mean and 85th percentile speeds versus the speed limit computed for each of the three calculation methods. The results showed that mean

Table 19. Descriptive statistics for mean and 85th percentile speeds versus speed limit.

Speed Limit (mph)	Number of Sites	Number of Vehicles Sampled	Mean Speeds (mph)			85th Percentile Speeds (mph)		
			Pooled	Average of Sites	Predicted by Regression	Pooled	Average of Sites	Predicted by Regression
25	4	368	30.15	30.69	28.40	35.00	35.61	33.48
30	4	325	32.32	32.97	32.53	38.07	37.95	37.91
35	4	320	37.94	37.52	36.66	42.00	41.71	42.33
40	16	719	39.46	39.77	40.79	46.49	46.51	46.75
45	17	893	44.12	43.79	44.92	51.14	49.86	51.18
50	10	644	49.88	50.13	49.06	57.65	57.06	55.60
55	5	363	56.89	55.34	53.19	62.94	61.14	60.02

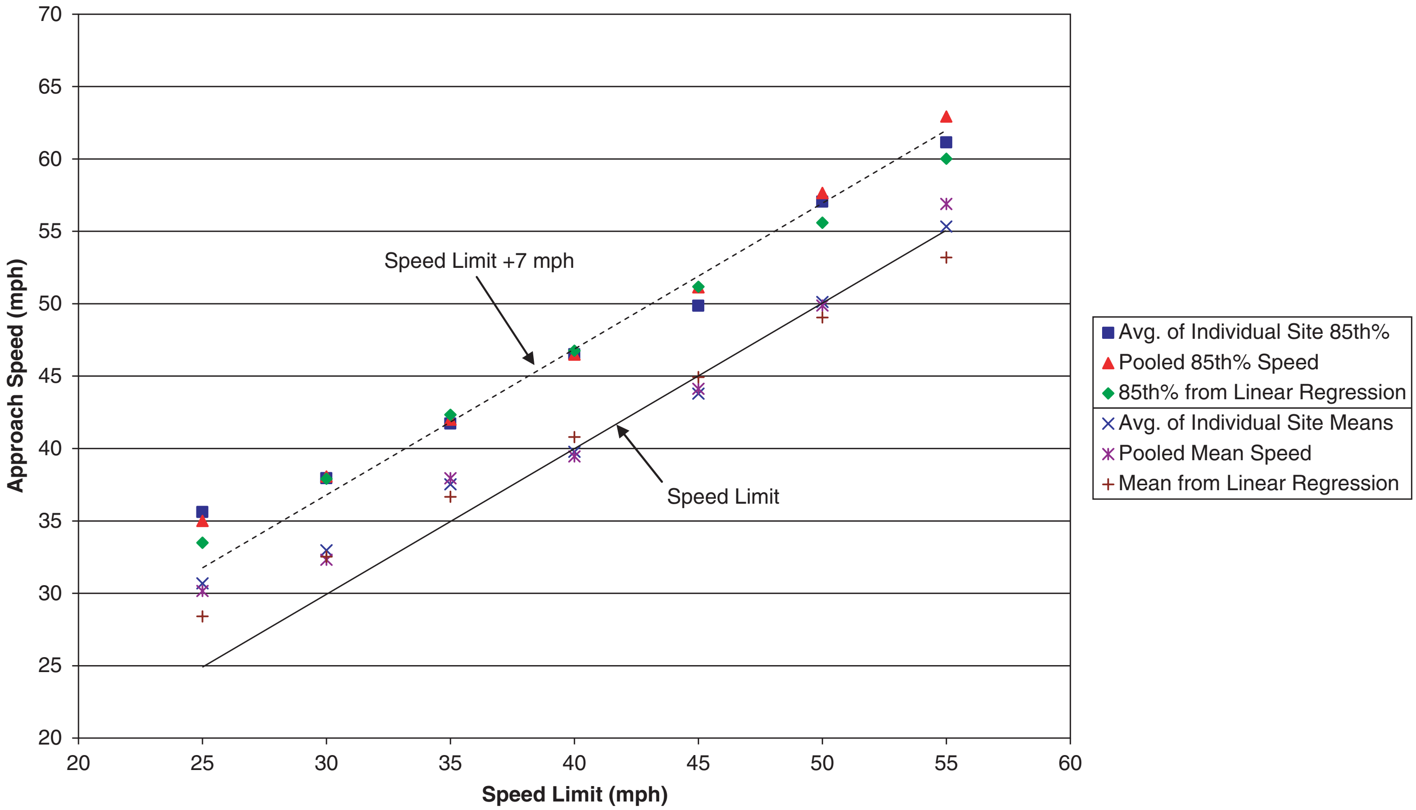

Figure 17. Mean and 85th percentile speed versus speed limit: comparison of calculation methods.

speeds typically exceeded the speed limit at speed limits of 35 mph and below and were approximately equal to the speed limit at speed limits between 40 mph and 55 mph.

Table 20 displays the difference between both the mean and 85th percentile speeds and the speed limit for each of the three calculation methods. When the speed data were pooled together within the respective speed limit category, the overall mean speed was 0.84 mph greater than the speed limit, while the overall 85th percentile speed was 7.45 mph greater than the speed limit. When the speed statistics were computed on a site-by-site basis, the mean speeds were, on average, 0.39 mph greater than the speed limit, while the 85th percentile speeds were, on average, 6.49 mph greater than the speed limit.

The results indicate that for practical purposes, the speed limit provides a good estimate of the *mean speed* of free-flowing vehicles approaching a signalized intersection. However, at nearly all locations, the *85th percentile speed* was found to exceed the speed limit—in some cases by greater than 10 mph. Thus, it can be concluded that the speed limit by itself typically does not provide an accurate estimate of the 85th percentile speed.

Table 20. Difference between mean and 85th percentile speeds and the speed limit.

Speed Limit (mph)	Mean Speed Minus Speed Limit (mph)			85th Percentile Speed Minus Speed Limit (mph)		
	Pooled	Average of Sites	Predicted by Regression	Pooled	Average of Sites	Predicted by Regression
25	5.15	5.69	3.40	10.00	10.61	8.48
30	2.32	2.97	2.53	8.07	7.95	7.91
35	2.94	2.52	1.66	7.00	6.71	7.33
40	-0.54	-0.23	0.79	6.49	6.51	6.75
45	-0.88	-1.21	-0.08	6.14	4.86	6.18
50	-0.12	0.13	-0.94	7.65	7.06	5.60
55	1.89	0.34	-1.81	7.94	6.14	5.02
OVERALL	0.84	0.39	0.79	7.45	6.49	6.75

With respect to timing of the yellow change and red clearance intervals, it was desirable to keep any operating speed recommendations as simple as possible—preferably based on the speed limit at the site plus a constant value. As shown in Table 20 and Figure 17, the 85th percentile approach speed for free-flowing vehicles can be accurately predicted for all speed limits except 25 mph based on the speed limit plus 7 mph. At speed limits of 25 mph, the 85th percentile speed is better predicted by the speed limit plus 10 mph. Thus, in lieu of field-measured speed data, it is recommended that the approach speed limit plus 7 mph be utilized as a rule-of-thumb estimate for the 85th percentile speed for purposes of timing of the yellow change and red clearance intervals for through vehicles.

Left-Turning Vehicles

The approach speeds for free-flowing left-turning vehicles were also measured at a select number of sites and compared to both the speed limit on the approach and the speeds of through-moving vehicles on the approach. Left-turning speeds were measured at approximately the same upstream location as speeds for through-moving vehicles. Thus, left-turn approach speeds were measured farther upstream at locations with higher speed limits. The subject vehicles were typically positioned in the leftmost through lane during the speed measurement and generally merged into the left-turn lane shortly after the speed measurement was made. Speeds were also obtained for vehicles already positioned in the left-turn lane and for vehicles that were braking before or within the speed measurement zone.

Speeds of left-turning vehicles were sampled from a total of 19 sites selected from the five states used in the study. The speed limits at the sites ranged from 40 to 55 mph. Locations with speed limits of 35 mph and lower were not included, as the operating speed for left-turning vehicles on low-speed approaches was assumed as being the same (or nearly so) as for through vehicles. The results of the left-turn approach speed data collection are shown in Table 21. The results show that the overall mean approach speeds for left-turning vehicles were 10.59 mph less than the posted speed limit. The overall 85th percentile left-turn approach speeds were 4.94 mph less than the posted speed limit. These values reinforced the assumption that drivers are typically preparing to decelerate in order to complete the left-turn maneuver. Thus, in lieu of field-measured speed data, it is recommended that the approach speed limit minus 5 mph be utilized as a rule-of-thumb estimate for the 85th percentile speed for purposes of timing of the yellow change interval for left-turning vehicles.

Speeds were not measured during the completion of the left-turn movement, as these speeds are limited by the geometry of the turning movement. However, based on the AASHTO horizontal curve design speed equation (*37*), the average left-turn design speed for the study sites was calculated as 16.3 mph with an 85th percentile value of 18.5 mph. These design speed values are considered conservative based on the design side-friction factors provided by AASHTO. Thus, in lieu of field-measured speed data, it is recommended that 20 mph be utilized as a rule-of-thumb estimate for the 85th percentile speed for purposes of timing of the red clearance interval for left-turning vehicles regardless of the approach speed limit.

Results: Intersection Entry Delay

The red clearance interval is traditionally intended to provide enough time for a driver crossing the stop line at the last moment of the yellow change interval to safely clear the intersection, reach the crosswalk, or clear the crosswalk. Current practice typically does not adjust the duration of the red interval to account for intersection entry delay by vehicles on the conflicting approach, although this has been suggested in previous guides (*10*).

The setback of the stop line creates a spatial buffer, thereby delaying entry into the intersection at the onset of the green signal indication. Additionally, there typically exists a certain amount of start-up delay for stopped drivers to begin forward movement at the onset of the green signal indication. A driver's start-up delay is dependent on several factors, including physical capabilities, level of distraction, and detection

Table 21. Mean and 85th percentile approach speeds of left-turning vehicles versus speed limit.

Speed Limit (mph)	Number of Sites	Number of Vehicles Sampled	Mean Speed (mph)	Mean Speed Minus Speed Limit (mph)	85th Percentile Speed (mph)	85th Percentile Speed Minus Speed Limit (mph)
40	3	90	31.85	-8.15	37.49	-2.51
45	11	332	34.64	-10.36	39.91	-5.09
50	3	88	36.61	-13.39	44.02	-5.98
55	2	60	43.61	-11.39	50.90	-4.10
OVERALL	19	570		-10.59		-4.94

of a potential red-light runner. The start-up delay provides an additional amount of buffer time that delays the entry of vehicles from adjacent approaches into the path of potential red-light running vehicles.

In order to quantify intersection entry delay, vehicular data were extracted from the videos for a sample of intersections. Twenty (20) intersection approaches evenly distributed among four of the study states were randomly selected for use in this evaluation. The intersections included a broad array of conditions, including traffic volumes, speed limits, intersection configurations, and stop line setbacks.

The videos for a sample of 20 randomly selected signal cycles were reviewed for each of the 20 intersections utilized in this evaluation. The following information was extracted for the initial vehicle in the queue on the adjacent approach for each of the selected signal cycles:

- Start-Up Delay, measured as the time after the end of the red clearance interval for the primary approach (i.e., the start of green on the adjacent opposing approach) when forward movement began for the subject vehicle. Information pertaining to whether or not the vehicle was stopped or rolling forward at the start of the green was also recorded. The start-up delay was recorded as 0 if the vehicle was already moving forward at the start of green.
- Total Intersection Entry Delay, measured as the start-up delay plus the incremental time for the front of the subject vehicle to reach the near edge of the closest conflicting travel lane.

All time-related data were determined based on the timestamp shown in the video review window. Data were recorded both for vehicles that were completely stopped and vehicles that were rolling forward at the onset of green. Both through-moving vehicles and turning vehicles were included in the sample. An example of the intersection entry delay assessment is shown in Figure 18.

Two datasets were created based on whether or not the vehicle remained completely stopped until after the onset of the green. The two datasets included the following:

- Vehicles that had not begun to move until after the start of green (n = 340), and
- A combination of vehicles that had not begun to move until after the start of green (n = 340) and vehicles that had begun moving forward prior to the green (n = 52).

The average start-up delay and total intersection entry delay times were then computed for each site for both datasets and are shown in Table 22. The overall averages were computed based on the average of the 20 individual site means, with the results shown below and in Table 22:

- Start-up delay after start of green:
  - Stopped vehicles: 1.22 s and
  - Stopped and rolling vehicles: 1.10 s.
- Total intersection entry delay after start of green:
  - Stopped vehicles: 4.38 s and
  - Stopped and rolling vehicles: 4.10 s.

As expected, the combined data set showed both shorter start-up delays and shorter intersection entry delays compared to data for vehicles that remained stopped until after the start of green. This is because vehicles that were rolling forward at the start of green always had zero start-up delay and subsequently required less time to enter the intersection. Under most circumstances, considering a combination of both stopped vehicles and vehicles rolling forward at the start of green provides a more accurate representation of actual driver behavior.

Figure 18. Example intersection entry delay assessment.

Table 22. Start-up delay and total intersection entry delay after the start of green for first vehicle in queue.

Primary Approach	Adjacent Approach	State	Red (s)	Average Start-Up Delay After the Start of Green (s)		Average Total Time After the Start of Green to Enter Near Travel Lane (s)	
				Stopped Vehicles	Stopped and Rolling Vehicles	Stopped Vehicles	Stopped and Rolling Vehicles
EB Warren	Cass	MI	1	1.08	1.08	5.09	5.09
NB Woodward	Kirby	MI	2.5	1.42	0.98	4.87	4.32
SB Anthony Wayne	Kirby	MI	1	0.93	0.78	4.08	3.92
SB Carpenter	Center Valley	MI	1.4	1.18	1.04	4.64	4.31
WB Ellsworth	Carpenter	MI	2.3	1.18	1.14	4.27	3.86
EB Packard	Carpenter	MI	2	1.35	1.35	4.79	4.79
EB Central Florida	Westwood	FL	1	1.36	1.36	4.25	4.25
NB International	Central Florida	FL	1	1.73	1.70	4.40	4.26
SB Kirkman	Conroy	FL	2	1.48	1.48	4.77	4.73
NB SR 535	Hotel Plaza	FL	1	1.00	0.98	4.25	3.96
SB SR 535	Hotel Plaza	FL	1	1.23	1.07	3.72	3.23
NB Fullerton	Pathfinder	CA	1	0.58	0.58	4.09	4.07
WB Whittier	Atlantic	CA	0	1.13	1.10	4.46	4.33
WB Dyer	Pullman	CA	1	0.96	0.93	2.98	2.86
WB Ball	East	CA	1	0.88	0.83	3.40	3.23
NB Imperial	LaPalma	CA	1	1.55	1.53	4.93	4.63
EB Lee Jackson Hwy	Loudoun Pkwy	VA	3	1.23	1.10	4.81	4.24
EB Leesburg Pike	Countryside	VA	2	1.25	1.22	3.88	3.83
NB Fairfax Co Pkwy	West Ox Rd	VA	2	1.09	0.33	4.34	3.09
NB Fairfax Co Pkwy	Fox Mill Rd	VA	2	1.70	1.35	5.52	4.96
NUMBER OF VEHICULAR OBSERVATIONS				340	392	340	392
AVERAGE OF ALL SITES				1.22	1.10	4.38	4.10

CHAPTER 6

Discussion of Study Results

This chapter presents the authors' assessment of the findings of this study. Through the literature review, the survey of state and local agency practices, and the field data results, the authors provide rationale for the proposed guideline for timing of yellow change and red clearance intervals, which is found in Appendix A.

What Model Should Be Followed to Determine Yellow Change and Red Clearance Intervals?

As discussed in Chapter 3, several procedures or models have been proposed and adopted to determine yellow change and red clearance interval durations. Of the alternative methods, it is the authors' opinion that the kinematic equation is the preferred method for calculating the interval durations. The kinematic equation is based on principles of physics and intersection characteristics; therefore, it is the most defensible and adaptable method used in practice. The yellow change interval—based on PRT, approach speed, deceleration rate, and approach grade—provides enough time for a vehicle to comfortably decelerate to a stop. The red clearance interval—based on intersection width, vehicle length, and approach speed—provides additional time as a safety measure for a vehicle that has entered the intersection at the last moment of yellow to avoid conflict with traffic releasing from an adjacent opposing intersection approach. A nationwide state-of-the-practice survey found that the kinematic equation (or variation thereof) is the most widely utilized procedure for calculating yellow change and red clearance intervals.

The standard kinematic model has had few changes since its adoption in the ITE *Traffic Engineering Handbook* in 1965. A modification factor to accommodate approach grade was incorporated in the 1982 edition of the ITE *Manual of Traffic Signal Design* and has since been the proposed method in subsequent editions of the ITE *Traffic Engineering Handbook*. The application of the kinematic equation, particularly the allocation of the change interval duration between yellow and red, is dependent upon the state vehicle code.

Should Yellow Change and Red Clearance Interval Timing Practices Vary Based on State Vehicle Code?

There are two basic laws that apply to the meaning of the yellow signal indication and the legal driver behavior in response to its display: "permissive" and "restrictive." It is unclear whether drivers (and even law enforcement) are aware of the state vehicle code with respect to the yellow law. Furthermore, it is doubtful that drivers are aware of differing laws or alter their behavior when traveling between jurisdictions.

States following the "restrictive" yellow law will ideally provide the entire change and clearance duration (yellow + red) to the yellow interval for the signal timing to be in harmony with the vehicle code. From a human factors perspective, providing a longer yellow change interval for a prevailing approach speed generally encourages drivers to enter later during the yellow (*47*). In this situation, a driver may enter the intersection while the yellow signal is displayed, but not clear the intersection prior to the red signal being displayed—a violation of the "restrictive" yellow law. In addition, where a "permissive" yellow law provides a red clearance interval to allow additional time as a safety factor for drivers legally within the intersection to avoid conflict with traffic releasing from an adjacent opposing intersection approach, a "restrictive" yellow law does not necessarily provide this added factor of safety. Furthermore, many restrictive jurisdictions allocate the interval durations based on the typical permissive timings (i.e., yellow as the change interval and red as the clearance interval), thereby creating intervals that are inconsistent with the law. In light of these issues, it is the authors' recommendation that yellow change and red clearance intervals be calculated and implemented in accordance with the "permissive" yellow law, following the language of the national *Uniform Vehicle Code* (*16*). The authors also propose that states currently following the "restrictive" yellow law consider changing their vehicle code to follow the "permissive" yellow law to provide nationwide uniformity and better alignment with driver expectations.

What Are the Recommended Equation and Associated Values?

The findings of this study recommend the equations shown in Equations 12 and 13 be used to determine the yellow change and red clearance intervals, respectively:

$$Y = t + \frac{1.47V}{2a + 64.4g} \qquad \textbf{Equation 12}$$

$$R = \frac{W + L}{1.47V} - 1 \qquad \textbf{Equation 13}$$

Where:
- t = PRT (s),
- a = deceleration rate (ft/s^2),
- V = 85th percentile approach speed (mph),
- g = approach grade (percent divided by 100, negative for downgrade),
- W = intersection width measured from the back edge of the approaching movement stop line to the far side of the intersection as defined by the extension of the curb line or outside edge of the farthest travel lane (ft), and
- L = length of vehicle (ft).

The recommended input values for Equations 12 and 13 are provided in the sections that follow.

Yellow Change Interval

Perception-Reaction Time (t)

One second has been the suggested reaction time in the kinematic equation since the 1965 edition of the ITE *Traffic Engineering Handbook.* (This term has been identified as the PRT, brake-reaction time, or simply reaction time.) This constant recognizes that there is a period of time from the moment a driver sees the signal change from green to yellow to when the driver reacts. The reaction and associated time depends upon the driver's position relative to the intersection. The reaction can be (1) to stay at current speed, continuing into the intersection; (2) to accelerate, perceiving the necessity to increase speed to legally enter the intersection; or (3) to decelerate, intending to stop.

Reaction time is a constant in the kinematic equation, having a direct one-to-one effect on the calculated interval. For example, if 1.3 seconds were used, then the calculated duration would be 0.3 seconds longer than if 1.0 seconds were used. This factor was examined for drivers who came to a stop in response to the onset of a yellow signal display. The mean value was found to be exactly 1.0 seconds with an 85th percentile value of 1.33 seconds. It was determined that a driver's reaction time and deceleration rate are dependent on one another. Slow-reacting drivers tend to compensate with greater deceleration rates whereas quick-reacting drivers tend to decelerate more comfortably. This point is thoroughly addressed by Parsonson (*58*) and was confirmed in the field study performed as part of this research. This research also indicated that most temporal-related site factors had little to no impact on reaction time or deceleration rate. Thus, it is recommended to consider the mean or median value when selecting a reaction time and, as will be discussed, deceleration rate for calculating the yellow change interval. Consequently, the authors propose that 1.0 seconds be used for the value of reaction time in the kinematic equation.

Deceleration Rate (a)

Deceleration rate has the largest effect on the variance of the calculated change interval when applying the kinematic equation. Hence, determining the most appropriate value is important to calculate the yellow change interval. The length of the yellow change interval is inversely related to the deceleration rate (i.e., as deceleration rate decreases, the calculated duration increases).

The value of 15 ft/s^2 was assumed in ITE manuals until the 1982 ITE *Manual of Traffic Signal Design* and the 1982 ITE *Traffic Engineering Handbook,* when it was reduced to the currently assumed 10 ft/s^2. Parsonson and Santiago (*31*) asserted that 15 ft/s^2 was for emergency stopping distance and should not apply to change interval timing. The selected rate should allow for a "comfortable" deceleration upon seeing the yellow signal display within the dilemma zone.

This research found the mean deceleration rate to be 10.08 ft/s^2 with an 85th percentile value of 12.89 ft/s^2. These values varied as follows:

- Increased as approach speeds increase (i.e., faster drivers used greater deceleration);
- Decreased as travel times to the intersection at the start of yellow increase (i.e., drivers used lower deceleration when farther from the intersection); and,
- Increased as the brake-response times increase (i.e., slower-reacting drivers used greater deceleration rates).

As stated previously, reaction time and deceleration rate were found to be directly correlated with each other with minimal impact from temporal-related site factors. Thus, it is recommended to consider the mean or median value when selecting a reaction time and deceleration rate for timing the yellow change interval. Consequently, the authors propose that a 10 ft/s^2 deceleration rate be used in the kinematic equation.

Approach Speed (V)

The speed variable appears in the equations for both the yellow change and red clearance intervals. The ITE *Traffic Engineering Handbook* uses the term "design speed," but does not elaborate on what that term means when applied to the equation. Unless the signal timing is being prepared for a traffic signal on a new road where the design speed is known, this speed metric is unavailable. Also, the actual operating speed, specifically the 85th percentile speed, is typically different than the design speed.

Previous editions of the ITE *Traffic Engineering Handbook* and other national resource publications recommend the 85th percentile speed be used when calculating these intervals; however, in some, the speed limit is used. Selecting the 85th percentile as the design specification has ample precedent in traffic engineering. The 85th percentile speed is based on the premise that the vast majority of drivers will select a speed that is reasonable, safe, and prudent for a given road. The 85th percentile approach speed is the preferred variable measurement to use in the kinematic equation, as it captures 85 percent of the drivers traveling at or below the value. Using the 85th percentile speed requires that a speed study be conducted to establish prevailing conditions, which is not always practical. Technical procedures for determining the 85th percentile speed are presented in detail in the ITE *Manual of Transportation Engineering Studies* (*59*).

This research found that, in practice, speed limit is more often used because it is more easily identified. Determining the appropriate speed limit is usually based on the 85th percentile speed observed from a speed study. If this were the case, then using the speed limit would be both practical and appropriate. However, the 85th percentile speed is typically higher than the speed limit, as was demonstrated by the data.

The findings of the field study showed that the overall 85th percentile speed was, on average, 7.45 mph greater than the posted speed limit. On a site-by-site basis, the 85th percentile speeds were, on average, 6.49 mph greater than the posted speed limit. Thus, it was concluded that the speed limit typically does not provide an accurate estimate of the 85th percentile speed.

With respect to timing practice, it is desirable to keep operating speed recommendations as simple as possible—preferably based on the posted speed limit plus a constant value. Thus, in lieu of field-measured speed data, the authors propose using the approach speed limit plus 7 mph as a rule-of-thumb estimate for the 85th percentile approach speed used to calculate the yellow change interval for through movements.

The field study also investigated approach speeds for left-turning vehicles. Left-turning drivers must decelerate to safely navigate the turn, which may not be a direct response to the yellow signal indication. Approaches with posted speed limits of 40 mph to 50 mph showed the overall mean left-turn approach speed to be 10.59 mph less than the posted speed limit. The overall 85th percentile left-turn approach speed was 4.94 mph less than the posted speed limit. Locations with posted speed limits of 35 mph or less were not investigated, as the operating speed for left-turning vehicles on low-speed approaches was assumed as being the same (or nearly so) as for through vehicles. For all approaches, the authors propose using the approach speed limit minus 5 mph to calculate the left-turn yellow change interval duration.

Grade (g)

Grade was introduced to the kinematic equation in 1982 and has been used ever since. Literature indicates that for every 1 percent upgrade, the duration of the calculated yellow change interval is decreased by 0.1 seconds. Conversely, for every 1 percent downgrade, the duration of the calculated yellow change interval is increased by 0.1 seconds.

The results of the field study showed grade to have an impact on reaction times and deceleration rates. Steep downgrades had an impact on reaction time, as 84 percent of drivers began braking prior to the onset of the yellow change interval, helping to keep deceleration rates at comfortable levels. Sites with upgrades greater than 3 percent exhibited deceleration rates that were much higher than on level grade, whereas sites with downgrades greater than 3 percent exhibited deceleration rates that were much lower than on level grade.

The authors do not propose that grade-related adjustments be made to reaction time and deceleration rate. However, approach grade should continue to be accounted for explicitly in the kinematic equation when calculating the yellow change interval. As discussed later in this chapter, the ±0.1 seconds will have an effect on the implementation of the change interval timings. The authors also propose that, as a general rule, the grade measurement be taken at the distance corresponding to the upper boundary of the dilemma zone (i.e., approximately 5.0 seconds upstream of the stop line) based on the approach speed limit plus 7 mph.

Red Clearance Interval

Approach Speed (V)

Some timing procedures or methods assume a different approach speed when calculating the red clearance interval. Rather than using the same 85th percentile approach speed as for the yellow change interval, the 15th percentile or even 10th percentile approach speed is suggested (*30*). Slower traveling vehicles will be exposed to or create potential conflict longer than faster traveling vehicles and therefore would require more red clearance time. However, applying the lower speed threshold would be overly conservative and inappropriate. If the yellow change interval is assuming the 85th percentile vehicle, then the same vehicle should be used for determining the red clearance interval. Drivers entering the intersection after the yellow signal has been displayed for several seconds are not likely to reduce their speed. Therefore, the authors propose using the same speed value to calculate the red clearance interval for through vehicles that was used to calculate the yellow change interval.

Turning vehicle speeds within the intersection were not collected and therefore data cannot be provided. However, based on the AASHTO horizontal curve design speed calculation equation (*37*), the average left-turn design speed for the study sites was calculated as 16.3 mph with an 85th percentile value of 18.5 mph. The literature suggests that left-turn maneuvers are typically performed at speeds between 15 and 25 mph, depending on the turning radius. Therefore, the authors recommend using a speed of 20 mph to calculate the left-turn red clearance interval duration regardless of the approach speed limit.

Length of Vehicle (L)

The length of the vehicle used in the calculation of the red clearance interval typically ranged from 15 feet to 20 feet. AASHTO (*37*) specifies a design passenger car as 19 feet. Arguments have been made to consider larger vehicles in the timing calculations, such as single-unit trucks (30 feet) or intermediate semitrailers (55 feet). Considering larger vehicles would increase the duration of the red clearance interval to accommodate the additional length prior to conflicting traffic being released. However, conflicting vehicular traffic is obligated to yield the right-of-way to other vehicles legally in the intersection (*1*), which would include truck trailers. The length of the vehicle is irrelevant to this requirement. Therefore, the authors propose a vehicle length of 20 feet for calculation of the red clearance interval.

Intersection Width (W)

The most recent ITE *Traffic Engineering Handbook* (*8*) defines intersection width as the distance from the stop line to the far-side no-conflict point. Other national resource publications reviewed for this research suggest the following options, listed in order of shortest to longest distance:

1. Curb to curb (or the extension of the travel edges of the conflicting roads if there are no curbs);
2. Near-side stop line to the middle of the first conflicting traffic lane;
3. Near-side stop line to the far edge of the last conflicting traffic lane;
4. Near-side stop line to far-side curb; or,
5. Near-side stop line to far side of the crosswalk, if one exists.

Note that the widths for Options 2 and 3 may be nearly the same depending upon the intersection geometric layout.

Channelized right-turn lanes would not have an impact on intersection width. At locations where a dedicated receiving lane is present, drivers can traverse the channelization under free-flow conditions and not be in conflict with through vehicles. Where a dedicated receiving lane is not present, drivers are controlled by a STOP or YIELD sign due to the potential for conflict with through vehicles and must abide by right-of-way rules.

A conservative position would suggest that Option 5 be followed to calculate the red clearance interval, ensuring that pedestrians are not released until conflicting vehicles clear the crosswalk. A liberal position would argue that a pedestrian would (should) be cognizant of a vehicle approaching the crosswalk area and not enter into the path of the oncoming vehicle. In situations involving blind or visually impaired pedestrians, auditory cues are utilized to establish vehicle presence, direction of travel, and acceleration rate or speed (*60*). Using these cues, blind or visually impaired pedestrians would likely be aware of an oncoming vehicle and not enter the crosswalk. However, safely accommodating this user group is often a challenge, as intersection geometries, signal equipment, vehicle characteristics, and driver behaviors vary greatly.

Excluding Option 5, the next conservative approach would be Option 4 to ensure that potential conflicting vehicles are through the intersection. The more liberal approach would be Option 2 to accommodate conflicting vehicles from the stop line to the middle of the first conflict lane (likely the left-turn lane). The argument for this option is that drivers on the conflicting approach would see a vehicle in the intersection and are obligated by law to not enter until the vehicle has passed.

Clearing width with respect to the presence of crossings equipped with pedestrian signals should be considered on receiving lanes. As suggested above, a conservative approach to measuring intersection width would be to include the entire width of the crossing as well. As stated in the current *MUTCD* under Section 4E.02 Standard A (*1*) (in accordance with the Uniform Vehicle Code (*16*)):

> *A steady WALKING PERSON (symbolizing WALK) signal indication means that a pedestrian facing the signal indication is permitted to start to cross the roadway in the direction of the signal indication, possibly in conflict with turning vehicles. The pedestrian shall yield the right-of-way to vehicles lawfully within the intersection at the time that the WALKING PERSON (symbolizing WALK) signal indication is first shown.*

Although a pedestrian is legally obligated to yield the right-of-way to a vehicle lawfully in the intersection, engineers maintain that a factor of safety should be considered to completely clear any conflicting vehicles and eliminate the possibility of pedestrian-vehicle conflict. Pedestrians, like drivers, exhibit a start-up delay when reacting to the change of a pedestrian signal indication from DON'T WALK to WALK. Pedestrian start-up delay is defined as the time from when the WALK signal becomes illuminated until the pedestrian first steps off the curb. Studies have been conducted to quantify this delay. The FHWA publication *Older Pedestrian Characteristics for Use in Highway Design* (*61*) indicates mean pedestrian start-up delay times of 1.93 seconds and 2.48 seconds for younger and older pedestrians, respectively. The 85th percentile pedestrian start-up delay times are reported as 3.06 seconds and 3.76 seconds for younger and older pedestrians, respectively. The *Highway Capacity Manual (HCM)* (*62*) recommends a pedestrian start-up delay design value of 3 seconds.

Using the pedestrian start-up delay values, a pedestrian-vehicle conflict distance can be calculated based on vehicle approach speed. Assuming the smallest start-up value of 1.93 seconds for a younger pedestrian, a vehicle approaching at 25 mph posted speed limit plus 7 mph (total speed of 47 ft/s) will traverse approximately 91 feet during this time. Taking into consideration a 1 second reduction of the red clearance interval to account for vehicular start-up delay (as discussed next), the effective pedestrian start-up delay becomes 0.93 seconds. For the same approach speed conditions, a vehicle will traverse approximately 43 feet during this time. These conditions account for the "worst case" scenario (i.e., quickest pedestrian reaction time and slowest vehicular approach speed). Vehicles approaching at faster speeds will traverse longer distances in the same time period. However, for uniformity in timing practices, the worst case threshold will be considered and applied to all approach speeds. In addition, the presence of a pedestrian crossing should only be a matter of concern if pedestrian signals are present, as the pedestrian signal indicates when it is permissible to leave the curb. For crossings without pedestrian signals, the pedestrians are not prompted by the pedestrian indication and must determine on their own when it is safe to cross.

In consideration of the above discussion, the authors propose the width of the intersection be measured from the back/upstream edge of the approaching movement stop line to the far side of the intersection as defined by the extension of the curb line or outside edge of the farthest travel lane. A pedestrian crossing equipped with pedestrian signals on a receiving lane should not be considered unless the nearest crossing line is 40 feet or more from the extension of the farthest edge of the farthest conflicting traffic lane. If this condition exists, the intersection width should be measured from the back/upstream edge of the approaching movement stop line to the nearest pedestrian crossing line.

Note that for left-turn movement red clearance interval calculations, the authors propose the width of the intersection be measured as the length of the approaching vehicle turning path from the back/upstream edge of the approaching movement stop line to the far side of the intersection as defined by the extension of the curb line or outside edge of the farthest travel traffic lane. If a pedestrian crossing equipped with pedestrian signals is present on the receiving lane and is 40 feet or more from the extension of the farthest edge of the farthest conflicting traffic lane, then measure to the nearest pedestrian crossing line.

Reduction (-1 second)

Equation 13 includes a reduction of 1 second to the calculated red clearance interval. This is recommended to account for the delay that is typically exhibited by the lead vehicle waiting on the conflicting approach to react to the green signal display and begin moving forward. The field study evaluated start-up delay for a combined data set of stopped and rolling vehicles and determined an average value of 1.1 seconds. Similarly, the field study evaluated total intersection entry delay (start-up delay plus the incremental time for the front of a vehicle to reach the near edge of the closest conflicting travel lane) for a combined data set of stopped and rolling vehicles and determined an average value of 4.1 seconds. Based on these findings, it was concluded that a 1-second reduction should be included in determining the red clearance interval. This value is conservative versus a 4-second reduction, considering the additional factors that delay intersection entry. The reduction of 1 second from the red clearance interval is not unprecedented, as the ITE *Traffic Control Devices Handbook* (*10*) also recommended this practice.

Reducing the calculated red clearance interval by 1 second has the potential to create a Type I dilemma zone (by the traditional definition), where a driver may not be entirely clear of the intersection prior to the beginning of the next conflicting phase. This may be a concern for corridors with a progressive signal system, as drivers become familiar with signal coordination and enter an intersection at speed as the signal display turns to green (also referred to as "gaming"). However, drivers entering an intersection from an opposing approach under the green signal indication are obligated to yield the right-of-way to other traffic legally within the intersection. This applies regardless of change interval durations and the definition of a Type I dilemma zone. Essentially, the 1-second reduction redefines the Type I dilemma zone in terms of "intersection clearance," recognizing that an intersection need be only partially cleared and that the start-up delay and spatial buffers provide for further intersection clearance prior to potential conflict.

Appendix F analyzes the effect of the 1-second reduction on intersection clearance. Two speed scenarios were assumed: (1) the posted speed limit plus 7 mph (a generally accurate estimate of the 85th percentile approach speed) and (2) the posted speed limit (a common practitioner's estimate of the approach speed). The analysis shows that in all scenarios the clearing driver has exited the intersection by the end of the 1-second start-up delay. Hence, under these conditions, the opposing driver stopped at the adjacent approach may see a vehicle in his/her path (that has legally entered the intersection on yellow) on the onset of green. However, this vehicle will typically have cleared the conflict area before the opposing driver has even begun rolling forward and would be well beyond the conflict area by the time the opposing driver reaches the nearest conflict point.

Should There Be Minimum and Maximum Values for Yellow Change and Red Clearance Intervals?

The *MUTCD* provides guidance with regard to the minimum and maximum durations of the yellow change and red clearance intervals. For the yellow change interval, the minimum is 3 seconds and the maximum is 6 seconds. For the red clearance interval, the maximum is 6 seconds. The literature review did not reveal any support for these suggested values. Presumably, the rationale behind providing minimum and maximum values is to prevent extremely short or excessively long durations. Extremely short durations may not provide an adequate level of safety whereas excessively long durations are counterproductive to efficient intersection operations. When implementing yellow change and red clearance intervals, there is a trade-off between intersection safety and intersection operations.

The authors find no reason to suggest minimum or maximum values for the yellow change interval. For the red clearance interval, the authors suggest using a minimum of 1 second to provide additional time to vehicles legally within an intersection as a factor of safety prior to the release of conflicting traffic from an adjacent opposing intersection approach. Anything less than 1 second would seem unreasonable; intuitively, a fraction of a second is not a beneficial amount of time to provide any added safety factor. A maximum red clearance interval is not suggested, however. Driver behavior and intersection conditions that will result in long red intervals are atypical; however, if behaviors and conditions are such that the calculated red interval is excessively long, then engineering judgment should be used to determine the appropriate application.

Is the Red Clearance Interval Necessary?

According to the current edition of the *MUTCD* (*1*), the use of a red clearance interval is optional by the following statement:

> *When indicated by the application of engineering practices, the yellow change interval should be followed by a red clearance interval to provide an additional time before conflicting traffic movements, including pedestrians, are released.* (Section 4D.26)

Elsewhere within that section, the following statements (shown in italics) are made about the red clearance interval, which are followed by an explanation as interpreted by the authors:

> *When used, the duration of the red clearance interval shall be determined using engineering practices. (Standard)*

This implies that the value chosen should not be arbitrarily determined, but should follow accepted engineering practice.

> *Engineering practices for determining the duration of . . . red clearance interval(s) can be found in ITE's* "Traffic Control Devices Handbook" *and in ITE's* "Manual on Traffic Signal Design" *(Support)*

This support statement identifies the two ITE publications described above that provide "accepted" engineering practice. Both provide equations for determining the duration of the red clearance interval.

> *The duration(s) of . . . red clearance intervals shall be consistent with the determined values within the technical capabilities of the controller unit. (Standard)*

This statement would apply to older electro-mechanical controllers that have rotary dials for allocating portions of the cycle for different phases. These controllers cannot accommodate settings in tenths of a second that modern digital controllers can.

> *Except as provided in Paragraph 12, the duration of the red clearance interval shall not be decreased or omitted on a cycle-by-cycle basis within the same signal timing plan. (Standard)*

This would apply to adaptive controller units for which the time for the different phases within a cycle, including red, can be changed based on traffic demand. It is possible that the traffic engineer would develop an algorithm within the controller that would reduce or even eliminate the red clearance interval under periods of heavy traffic and/or low speeds. Under this standard, this option would not be permitted.

The duration of a red clearance interval may be extended from its predetermined value for a given cycle based upon the detection of a vehicle that is predicted to violate the red signal indication. (Option)

This statement considers the possibility of having an approach detection system and controller that can predict that a vehicle continuing at its detected speed will not clear the intersection before the onset of green for the conflicting movement, and as such, the red can be extended as necessary.

When an actuated signal sequence includes a signal phase for permissive/protected (lagging) left-turn movements in both directions, the red clearance interval may be shown during those cycles when the lagging left-turn signal phase is skipped and may be omitted during those cycles when the lagging left-turn signal phase is shown. (Option)

This provision recognizes that when left turns are being made under the protected phase, there is no reason to provide a red interval for the adjacent through vehicles because the opposing traffic will face a red signal while the left-turn phase continues. Correspondingly, if there is no left-turning traffic during the protected phase (lagging), then the need for a red clearance interval for through traffic (and left-turners during the permissive phase) remains.

The duration of . . . a red clearance interval may be different in different signal timing plans for the controller unit. (Option)

This would apply to cases in which there are multiple signal timing plans (e.g., peak-period, night, weekend, etc.) within the controller. Under periods of heavy volume (i.e., peak-period plan), the engineer may want to reduce or eliminate the time for the red clearance interval under the assumption that more green time is needed to serve the heavy flow. In this case, the engineer is placing a priority on operational efficiency rather than safety.

Except when clearing a one-lane, two-way facility or when clearing an exceptionally wide intersection, a red clearance interval should have a duration not exceeding 6 seconds. (Guidance)

This condition is stating that the red clearance interval should not exceed 6 seconds except for two conditions: (1) for a one-lane, two-way facility—presumably relating to alternating one-way work zones or alternating one-way bridges and tunnels where red clearance is based on the length of the one-way section so that a vehicle is not trapped prior to the release of the opposing direction traffic, and (2) when clearing an "exceptionally" wide intersection—presumably one that would require more than 6 seconds as determined by the equation for the red clearance interval (e.g., single-point diamond interchange).

Since the provision of a red clearance interval is considered a safety factor, one would assume that there is evidence that the provision of a red clearance interval reduces the occurrence of angle crashes. *A priori,* one would expect a reduction in right-angle crashes at signalized intersections where the red clearance interval is used. As reported in the literature review section of this report, previously published empirical studies have not consistently or definitively demonstrated long-term crash reductions associated with the installation of red clearance intervals. In light of that finding, the decision by a transportation agency to not provide red clearance intervals should not necessarily be interpreted as being detrimental to safety. However, the authors propose that the yellow change and red clearance intervals be calculated in accordance with the "permissive" yellow law; therefore, a red clearance interval is appropriate.

What Other Aspects Should Be Considered When Implementing Yellow Change and Red Clearance Intervals?

Rounding of Calculated Values

Modern digital traffic signal controllers are capable of programming values to one-tenth of a second (0.1 s) for any interval; therefore, the timings for the yellow change and red clearance intervals can be calculated in tenths of a second. Using Equations 12 and 13 to calculate the yellow change and red clearance interval durations, the resulting values should be rounded to the nearest 0.1 seconds. Values ending in 0.01 to 0.04 should be rounded down to the nearest tenth of a second whereas values ending in 0.05 to 0.09 should be rounded up to the nearest tenth of a second.

If an existing agency policy rounds these values to the nearest half-second (0.5 s), then the following methodology is suggested:

- Values ending in 0.0 to 0.1 should be rounded down to the nearest whole number;
- Values ending in 0.2, 0.3, and 0.4 should be rounded up to the half-second;
- Values ending in 0.6 should rounded down to the half-second; and,
- Values ending in 0.7, 0.8, and 0.9 should be rounded up to the nearest whole number.

Left-Turn Change Intervals

When calculating yellow change and red clearance intervals for left-turning vehicles, signal phasing will ideally be considered as follows:

- For protected-only left-turn movements, the yellow and red intervals shall be calculated for each approach and implemented as calculated. The intervals do not have to be the same duration for opposing approaches.
- For permissive-only left-turn movements, the yellow and red intervals shall be calculated for opposing approaches, including the through movements. The implemented intervals shall be the longest of the calculated values (left, through, or combination). The intervals shall be the same duration for the left-turn and through movements on opposing approaches to ensure that termination is concurrent.
- For protected/permissive left-turn movements, the yellow and red intervals shall be calculated and implemented as described above for the respective protected and permissive portions of the phase.

Proposed Guidelines

One of the precepts on the use of all traffic control devices is that they be applied uniformly so that drivers can expect to experience the same device and its operation throughout their travels, within and outside of their jurisdiction. This would pertain to the timing of the yellow change and red clearance intervals. To maintain this precept, the proposed guidelines strive to achieve national acceptance for a uniform practical application. The guidelines are succinct in scope and require little user interpretation. As such, they provide a solid framework based on research and accepted practice that can be easily adopted into state or local transportation agency practice.

While the guidelines are based on typical roadway and driver conditions, there may be instances when exceptions are necessary to accommodate unusual conditions. Under these circumstances, the engineer or practitioner may exercise “engineering judgment” to determine that the conditions warrant the use of other calculation or implementation practices than those presented in the guidelines. However, under typical roadway and driver conditions, drivers should expect that the duration of the yellow change and red clearance intervals will be calculated according to the recommended kinematic equation and its associated recommended values.

CHAPTER 7

Conclusions and Recommended Research

Conclusions

The duration of the yellow change and red clearance intervals has an impact on driver behavior and intersection safety. The survey results and the literature review confirmed that agencies responsible for change interval timing take a widely varied approach in their practices. It appears, however, that the ITE kinematic equation (or a variation thereof) is used by the highest percentage of state and local agencies and is commonly referred to in national publications used by the traffic engineering community.

The following conclusions are drawn from this research study:

Literature Review

- There are a variety of methods used to calculate yellow change and red clearance interval durations. For the yellow change interval, the methods include the kinematic equation, "rule-of-thumb," uniform value, stopping probability, combined kinematic and stopping probability, and modified kinematic equation for left-turn movements. For the red clearance interval, the methods include the kinematic equation, uniform value, conflict zone, and modified kinematic equation for left-turn movements.
- Deceleration rate and perception-reaction time have the largest and second largest effect on the calculated interval, respectively. The literature suggests a value of 10 ft/s^2 for deceleration rate and 1 second for perception-reaction time.
- The 85th percentile speed is suggested as the most appropriate measure of approach speed.
- Vehicle length is typically assumed as 20 feet.
- The literature suggests that guidance be strengthened for intersection width measurement practices.
- Grade has been suggested to inversely affect the duration of the yellow change interval by ±0.1 seconds for every ±1 percent change (i.e., a 1 percent upgrade would decrease the duration of the calculated yellow change interval by 0.1 seconds, and vice versa).
- Driver behavior is influenced by traffic speed and volume, signal timing and coordination, number of lanes (i.e., intersection width), vehicle type, age, and gender. Other factors include weather conditions, regional driving practices, level and/or type of enforcement, and cell phone use.
- Using the current ITE guidelines to calculate the duration of yellow change and red clearance intervals has been shown to reduce total crashes between 8 and 14 percent while reducing injury crashes by approximately 12 percent.
- Increasing the yellow change interval to the duration calculated by current ITE guidelines has been shown to reduce red-light running occurrences between 36 and 50 percent. Increasing the red clearance interval to the duration calculated by current ITE guidelines has not shown to increase red-light running events; however, the crash results associated with installing red clearance intervals at locations previously without are unclear.

State of Practice

- There is a lack of uniformity in determining the duration of yellow change and red clearance intervals. The use of varying procedures in which engineering judgment plays a significant role creates a lack of consistency for both minimum and maximum values.
- Survey respondents utilizing the ITE kinematic equation (or a variation thereof) are commonly using generally accepted values for PRT (1 second), deceleration rate (10 ft/s^2), and vehicle length (20 feet). However, greater variation is seen with respect to approach speed and intersection width.

Field Studies

- For 2,422 vehicles sampled, the mean brake-response (perception-reaction) time was found to be 1.00 seconds. This validates the generally accepted value used in the current ITE guidelines.

- For 2,458 vehicles sampled, the mean deceleration rate was found to be 10.08 ft/s^2. This validates the generally accepted value used in the current ITE guidelines.
- For 3,632 vehicles sampled, the mean approach speed typically exceeded the speed limit at locations with a posted speed limit of 35 mph and below. At locations with a posted speed limit of 40 mph and above, the mean approach speed was approximately equal to the speed limit. However, at nearly all locations, the 85th percentile approach speed was found to exceed the posted speed limit. For locations with speed limits of 30 mph or greater, the 85th percentile approach speed is accurately estimated based on the speed limit plus 7 mph. For locations with speed limits of 25 mph, the 85th percentile approach speed is accurately estimated based on the speed limit plus 10 mph. Therefore, speed limit in itself does not provide an accurate estimate of 85th percentile speed.
- For 570 left-turning vehicles sampled at locations with posted speed limit between 40 mph and 55 mph, the mean approach speed was found to be 10.59 mph less than the speed limit whereas the 85th percentile approach speed was found to be 4.94 mph less than the speed limit. This is expected, as drivers are typically decelerating to complete the turning maneuver.
- For 392 stopped or rolling vehicles on conflicting intersection approaches, the average start-up delay time was found to be 1.1 seconds. This value accounts for the time after the end of the red clearance interval for the primary approach (i.e., the start of green on the adjacent opposing intersection approach) when forward movement began for the subject vehicle. The average total intersection entry time was found to be 4.1 seconds. This value represents start-up delay plus the incremental time for the front of the subject vehicle to reach the near edge of the closest conflicting travel lane.

Entire Guideline Analysis

- The kinematic equation should be the basis for determining the yellow change and red clearance intervals. This method is based on principles of physics and intersection conditions. It is the most defensible and adaptable method used in practice.
- Brake-response (perception-reaction) time and deceleration rate were found to be directly correlated with each other, thereby confirming that drivers do not select these parameters independent of each other. Slow-reacting drivers tend to compensate with greater deceleration rates whereas quick-reacting drivers tend to decelerate more comfortably. Thus, it is recommended to select the same metric, such as the mean or median, when selecting a reaction time and deceleration rate. Therefore, a reaction time of 1.0 seconds and a deceleration rate of 10 ft/s^2 are most appropriate.
- Speed limit by itself was found to be an inaccurate estimate of 85th percentile speed. In lieu of field-measured speed data to determine 85th percentile approach speed, the findings of this study suggest it is appropriate to estimate this value for through free-flowing vehicles by adding 7 mph to the approach speed limit. For left-turning vehicles, this study suggests the 85th percentile approach speed is appropriately estimated by subtracting 5 mph from the approach speed limit. When calculating the red clearance interval, the speed estimation holds true for through free-flowing vehicles. However, for left-turning vehicles, this study suggests using 20 mph regardless of posted speed limit.
- The vehicle length is suggested to be 20 feet, the generally accepted value for passenger cars. Increasing the length to accommodate larger vehicles is not considered necessary.
- A reasonable assumption for intersection width is the distance from the back/upstream edge of the near-side stop line to the far side of the intersection as defined by the extension of the curb line or outside edge of the farthest travel lane. For left-turning vehicles, this measurement would be along the turning path.
- To account for start-up delay of opposing traffic, the duration of the calculated red clearance interval is suggested to be reduced by 1 second. Conflicting vehicles typically do not enter the intersection until several seconds after the start of the green interval. This does not conform to the traditional definition of a Type I dilemma zone; however, conflicting vehicles are obligated to yield the right-of-way to vehicles legally entering the intersection during the yellow change interval.

Recommended Research

The procedure of determining the duration of the yellow change and red clearance intervals has had extensive research dating back to at least 1970. The authors would like to think that this study will have addressed all outstanding issues to the traffic engineering community's satisfaction. Assuming there is agreement with and acceptance of the guidelines for timing of the yellow change and red clearance intervals by the traffic engineering community, there does not appear to be any justification for additional research into this issue, specifically the formulation of the equation and its associated parameter values.

However, the authors suggest further research of the safety impacts associated with implementing a red clearance interval. Under the "permissive" law, the red clearance interval is used as an added safety measure. As was discovered through previously published literature, the safety benefit of providing a red clearance interval has yet to be conclusively confirmed. The studies that have been conducted were found to have shortcomings with regard to methodologies. Hence, given the concern of the need for a red clearance, it is recommended that research be conducted to isolate how the provision of a red clearance interval (and its length) affects the safety performance of the intersection.

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APPENDIX A

Guidelines for Timing Yellow and All-Red Intervals at Signalized Intersections

BACKGROUND

The yellow change interval is the period of time following the green signal indication during which a yellow signal indication is displayed. The red clearance interval is the period of time that follows the yellow signal indication during which a red signal indication is displayed to all conflicting movements at an intersection. The yellow change interval and red clearance interval are collectively referred to as the change interval.

The purpose of the yellow change interval is to warn drivers of an impending change in the right-of-way assignment. The purpose of the red clearance interval is to provide additional time as a safety factor for a driver that legally entered the intersection at the very last instant of the yellow change interval to avoid conflict with traffic releasing from an adjacent opposing intersection approach.

CHANGE INTERVAL CALCULATION

The yellow change and red clearance intervals are calculated using the equations and associated parameters as presented in the following sections.

Yellow Change Interval

The yellow change interval (Y) is calculated using Equation A:

$$Y = t + \frac{1.47V}{2a + 64.4g} \qquad \textbf{Equation A}$$

Where:

t = PRT (s); *set at 1.0 seconds*

a = deceleration rate (ft/s^2); *set at 10 ft/s^2*

V = 85th percentile approach speed (mph)

g = approach grade (percent divided by 100, negative for downgrade)

The value recommended for PRT (t) is 1.0 second and for deceleration rate (a) is 10 ft/s^2. The value for the approach speed (V) is recommended as the 85th percentile speed determined under free-flow conditions. If the 85th percentile approach speed is available, then the yellow change interval is calculated directly from Equation A. Since the 85th percentile speed is typically not available, it can be assumed as the posted speed limit plus 7 mph, except for left-turn movements (as explained). Table A provides yellow change intervals for through movements based on typical roadway and driver conditions assuming the posted speed limit plus 7 mph for grades in the range of ±4 percent.

Table A. Yellow Change Interval (seconds) by Approach Speed Limit and Grade

Posted Speed Limit (mph)*	Grade (%) -4	-2	0	2	4
25	3.7	3.5	3.4	3.2	3.1
30	4.1	3.9	3.7	3.6	3.4
35	4.5	4.3	4.1	3.9	3.7
40	5.0	4.7	4.5	4.2	4.1
45	5.4	5.1	4.8	4.6	4.4
50	5.8	5.5	5.2	4.9	4.7
55	6.2	5.9	5.6	5.3	5.0

*Yellow change intervals calculated using 85th percentile approach speed estimation of posted speed limit +7 mph

Red Clearance Interval

The red clearance interval (R) is calculated using Equation B:

$$R = \frac{W+L}{1.47V} - 1 \qquad \textbf{Equation B}$$

Where:

W = intersection width measured from the back/upstream edge of the approaching movement stop line to the far side of the intersection as defined by the extension of the curb line or outside edge of the farthest travel lane (ft)

L = length of vehicle (ft); *set at 20 feet*

V = 85th percentile approach speed (mph)

The width of the intersection (W) should be measured from the back/upstream edge of the stop line to the far-side intersection limit as determined by the extension of the curb line or outside edge of the farthest travel lane. A pedestrian crossing equipped with pedestrian signals on a receiving lane should not be considered unless the nearest crossing line is 40 feet or more from the extension of the farthest edge of the farthest conflicting traffic lane. If this condition exists, the intersection width should be measured from the back/upstream edge of the approaching movement stop line to the nearest pedestrian crossing line. The length of the vehicle (L) should be assumed as 20 feet. The same approach speed value used to calculate the yellow change interval should be used to calculate the red clearance interval, except for left-turn movements (as explained). The reduction of 1 second is to account for the start-up delay typically incurred by a driver stopped on a conflicting approach to react to a green signal indication and proceed forward.

The following provisions apply for specifying the duration of a calculated red clearance interval:

- If the calculated red clearance interval is less than or equal to 1.0 seconds, then the minimum implemented duration should be 1.0 seconds.

- If the calculated red clearance interval is greater than 1.0 seconds, then the implemented duration should be as calculated.

For Left-Turn Movements

Yellow change and red clearance intervals for left-turn movements should be calculated using Equations A and B with the following modified parameters:

Yellow Change Interval

V = approach speed (mph); *should be set at the approach speed limit minus 5 mph*

Red Clearance Interval

W = length of the approaching vehicle turning path measured from the back/upstream edge of the approaching movement stop line to the far side of the intersection as defined by the extension of the curb line or outside edge of the farthest travel lane (ft)*

V = approach speed (mph); *should be set at 20 mph regardless of the approach speed limit*

*A pedestrian crossing equipped with pedestrian signals on a receiving lane should not be considered unless the nearest crossing line is 40 feet or more from the extension of the farthest edge of the farthest conflicting traffic lane. If this condition exists, the intersection width should be measured from the back/upstream edge of the approaching movement stop line to the nearest pedestrian crossing line.

When calculating yellow change and red clearance intervals for left-turning vehicles, signal phasing should be considered as follows:

- For protected-only left-turn movements, the yellow and red intervals shall be calculated for each approach and implemented as calculated. The intervals do not have to be the same duration for opposing approaches.
- For permissive-only left-turn movements, the yellow and red intervals shall be calculated for opposing approaches, including the through movements. The implemented intervals shall be the longest of the calculated values (left, through, or combination). The intervals shall be the same duration for the left-turn and through movements on opposing approaches to ensure that termination is concurrent.
- For protected/permissive left-turn movements, the yellow and red intervals shall be calculated and implemented as described above for the respective protected and permissive portions of the phase.

OTHER CONSIDERATIONS

Grade Measurement

If a measurement of approach grade is required, as a general rule, it should be taken at the distance corresponding to the upper boundary of the dilemma zone (i.e., approximately 5.0 seconds upstream of the stop line) based on the approach speed limit plus 7 mph.

Unusual Conditions

While the guidelines are based on typical roadway and driver conditions, there may be instances when exceptions are necessary to accommodate unusual conditions. Under these circumstances, the engineer or practitioner may exercise “engineering judgment” to determine that the conditions warrant the use of other calculation or implementation practices than those presented in the guideline. However, under typical roadway and driver conditions, drivers should expect that the duration of the yellow change and red clearance intervals will be calculated according to the recommended kinematic equation and its associated recommended values.

Rounding

Modern digital traffic signal controllers are capable of programming values to one-tenth of a second (0.1 s) for any interval; therefore, the timings for the yellow change and red clearance intervals can be calculated in tenths of a second. Using Equations A and B to calculate the yellow change and red clearance interval durations, the resulting values should be rounded to the nearest 0.1 seconds. Values ending in 0.01 to 0.04 should be rounded down to the nearest tenth of a second whereas values ending in 0.05 to 0.09 should be rounded up to the nearest tenth of a second.

If an existing agency policy rounds change interval values to the nearest half-second (0.5 s), then the following methodology is suggested:

- Values ending in 0.0 to 0.1 should be rounded down to the nearest whole number;
- Values ending in 0.2, 0.3, and 0.4 should be rounded up to the half-second;
- Values ending in 0.6 should rounded down to the half-second; and,
- Values ending in 0.7, 0.8, and 0.9 should be rounded up to the nearest whole number.

APPENDIX B

Relevant *MUTCD* Sections

Section 4D.04 Meaning of Vehicular Signal Indications

Support:
01 The “Uniform Vehicle Code” (see Section 1A.11) is the primary source for the standards for the meaning of vehicular signal indications to both vehicle operators and pedestrians as provided in this Section, and the standards for the meaning of separate pedestrian signal head indications as provided in Section 4E.02.

02 The physical area that is defined as being “within the intersection” is dependent upon the conditions that are described in the definition of intersection in Section 1A.13.

Standard:
03 **The following meanings shall be given to highway traffic signal indications for vehicles and pedestrians:**

A. Steady green signal indications shall have the following meanings:

1. Vehicular traffic facing a CIRCULAR GREEN signal indication is permitted to proceed straight through or turn right or left or make a u-turn movement except as such movement is modified by lane-use signs, turn prohibition signs, lane markings, roadway design, separate turn signal indications, or other traffic control devices. Such vehicular traffic, including vehicles turning right or left or making a u-turn movement, shall yield the right-of-way to:

(a) Pedestrians lawfully within an associated crosswalk, and

(b) Other vehicles lawfully within the intersection.

In addition, vehicular traffic turning left or making a U-turn movement to the left shall yield the right-of-way to other vehicles approaching from the opposite direction so closely as to constitute an immediate hazard during the time when such turning vehicle is moving across or within the intersection.

2. Vehicular traffic facing a GREEN ARROW signal indication, displayed alone or in combination with another signal indication, is permitted to cautiously enter the intersection only to make the movement indicated by such arrow, or such other movement as is permitted by other signal indications displayed at the same time. Such vehicular traffic, including vehicles turning right or left or making a u-turn movement, shall yield the right-of-way to:

(a) Pedestrians lawfully within an associated crosswalk, and

(b) Other vehicles lawfully within the intersection.

3. Pedestrians facing a CIRCULAR GREEN signal indication, unless otherwise directed by a pedestrian signal indication or other traffic control device, are permitted to proceed across the roadway within any marked or unmarked associated crosswalk. The pedestrian shall yield the right-of-way to vehicles lawfully within the intersection or so close as to create an immediate hazard at the time that the green signal indication is first displayed.

4. Pedestrians facing a GREEN ARROW signal indication, unless otherwise directed by a pedestrian signal indication or other traffic control device, shall not cross the roadway.

B. Steady yellow signal indications shall have the following meanings:

1. Vehicular traffic facing a steady CIRCULAR YELLOW signal indication is thereby warned that the related green movement or the related flashing arrow movement is being terminated or that a steady red signal indication will be displayed immediately thereafter when vehicular traffic shall not enter the intersection. The rules set forth concerning vehicular operation under the movement(s) being terminated shall continue to apply while the steady CIRCULAR YELLOW signal indication is displayed.

2. Vehicular traffic facing a steady YELLOW ARROW signal indication is thereby warned that the related GREEN ARROW movement or the related flashing arrow movement is being terminated. The rules set forth concerning vehicular operation under the movement(s) being terminated shall continue to apply while the steady YELLOW ARROW signal indication is displayed.

3. Pedestrians facing a steady CIRCULAR YELLOW or YELLOW ARROW signal indication, unless otherwise directed by a pedestrian signal indication or other traffic control device shall not start to cross the roadway.

C. Steady red signal indications shall have the following meanings:

1. Vehicular traffic facing a steady CIRCULAR RED signal indication, unless entering the intersection to make another movement permitted by another signal indication, shall stop at a clearly marked stop line; but if there is no stop line, traffic shall stop before entering the crosswalk on the near side of the intersection; or if there is no crosswalk, then before entering the intersection; and shall remain stopped until a signal indication to proceed is displayed, or as provided below.

Except when a traffic control device is in place prohibiting a turn on red or a steady RED ARROW signal indication is displayed, vehicular traffic facing a steady CIRCULAR RED signal indication is permitted to enter the intersection to turn right, or to turn left from a one-way street into a one-way street, after stopping. The right to proceed with the turn shall be subject to the rules applicable after making a stop at a STOP sign.

2. Vehicular traffic facing a steady RED ARROW signal indication shall not enter the intersection to make the movement indicated by the arrow and, unless entering the intersection to make another movement permitted by another signal indication, shall stop at a clearly marked stop line; but if there is no stop line, before entering the crosswalk on the near side of the intersection; or if there is no crosswalk, then before entering the intersection; and shall remain stopped until a signal indication or other traffic control device permitting the movement indicated by such RED ARROW is displayed.

When a traffic control device is in place permitting a turn on a steady RED ARROW signal indication, vehicular traffic facing a steady RED ARROW signal indication is permitted to enter the intersection to make the movement indicated by the arrow signal indication, after stopping. The right to proceed with the turn shall be limited to the direction indicated by the arrow and shall be subject to the rules applicable after making a stop at a STOP sign.

3. Unless otherwise directed by a pedestrian signal indication or other traffic control device, pedestrians facing a steady CIRCULAR RED or steady RED ARROW signal indication shall not enter the roadway.

Section 4D.26 Yellow Change and Red Clearance Intervals

Standard:

01 **A steady yellow signal indication shall be displayed following every CIRCULAR GREEN or GREEN ARROW signal indication and following every flashing YELLOW ARROW or flashing RED ARROW signal indication displayed as a part of a steady mode operation. This requirement shall not apply when a CIRCULAR GREEN, a flashing YELLOW ARROW, or a flashing RED ARROW signal indication is followed immediately by a GREEN ARROW signal indication.**

02 **The exclusive function of the yellow change interval shall be to warn traffic of an impending change in the right-of-way assignment.**

03 **The duration of the yellow change interval shall be determined using engineering practices.**

Support:
04 Section 4D.05 contains provisions regarding the display of steady CIRCULAR YELLOW signal indications to approaches from which drivers are allowed to make permissive left turns.

Guidance:
05 *When indicated by the application of engineering practices, the yellow change interval should be followed by a red clearance interval to provide additional time before conflicting traffic movements, including pedestrians, are released.*

Standard:
06 **When used, the duration of the red clearance interval shall be determined using engineering practices.**

Support:
07 Engineering practices for determining the duration of yellow change and red clearance intervals can be found in ITE's "Traffic Control Devices Handbook" and in ITE's "Manual of Traffic Signal Design" (see Section 1A.11).

Standard:
08 **The durations of yellow change intervals and red clearance intervals shall be consistent with the determined values within the technical capabilities of the controller unit.**

09 **The duration of a yellow change interval shall not vary on a cycle-by-cycle basis within the same signal timing plan.**

10 **Except as provided in Paragraph 12, the duration of a red clearance interval shall not be decreased or omitted on a cycle-by-cycle basis within the same signal timing plan.**

Option:
11 The duration of a red clearance interval may be extended from its predetermined value for a given cycle based upon the detection of a vehicle that is predicted to violate the red signal indication.

12 When an actuated signal sequence includes a signal phase for permissive/protected (lagging) left-turn movements in both directions, the red clearance interval may be shown during those cycles when the lagging left-turn signal phase is skipped and may be omitted during those cycles when the lagging left-turn signal phase is shown.

13 The duration of a yellow change interval or a red clearance interval may be different in different signal timing plans for the same controller unit.

Guidance:
14 *A yellow change interval should have a minimum duration of 3 seconds and a maximum duration of 6 seconds. The longer intervals should be reserved for use on approaches with higher speeds.*

15 *Except when clearing a one-lane, two-way facility (see Section 4H.02) or when clearing an exceptionally wide intersection, a red clearance interval should have a duration not exceeding 6 seconds.*

Standard:
16 **Except for warning beacons mounted on advance warning signs on the approach to a signalized location (see Section 2C.36), signal displays that are intended to provide a "pre-yellow warning" interval, such as flashing green signal indications, vehicular countdown displays, or other similar displays, shall not be used at a signalized location.**

Support:
17 The use of signal displays (other than warning beacons mounted on advance warning signs) that convey a "pre-yellow warning" have been found by research to increase the frequency of crashes.

APPENDIX C

Definition of Yellow Signal for Vehicles by State

States	Definition of yellow signal (for vehicles)
Alabama (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
Alaska (P)	*No specific information available, assume Uniform Vehicle Code as default.*
Arizona (P)	Vehicular traffic facing a steady yellow signal is warned by the signal that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Arkansas (P)	Vehicular traffic facing the signal is warned that the red or "STOP" signal will be exhibited immediately thereafter, and vehicular traffic shall not enter the intersection when the red or "STOP" signal is exhibited.
California (P)	A driver facing a steady circular yellow or yellow arrow signal is, by that signal, warned that the related green movement is ending or that a red indication will be shown immediately thereafter.
Colorado (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
Connecticut*	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter, when vehicular traffic shall stop before entering the intersection unless so close to the intersection that a stop cannot be made in safety.
Delaware (P)	Vehicular traffic facing the circular yellow signal is thereby warned that a red signal for the previously permitted movement will be exhibited immediately thereafter.
Florida (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Georgia (P)	Traffic, except pedestrians, facing a steady CIRCULAR YELLOW or YELLOW ARROW signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Hawaii (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Idaho (P)	A driver facing a steady circular yellow or yellow arrow signal is being warned that the related green movement is ending, or that a red indication will be shown immediately after it.

States	Definition of yellow signal (for vehicles)
Illinois (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
Indiana (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is warned that the related green movement is being terminated and that a red indication will be exhibited immediately thereafter.
Iowa*	A "steady circular yellow" or "steady yellow arrow" light means vehicular traffic is warned that the related green movement is being terminated and vehicular traffic shall no longer proceed into the intersection and shall stop. If the stop cannot be made in safety, a vehicle may be driven cautiously through the intersection.
Kansas (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Kentucky (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Louisiana	Vehicular traffic facing a steady yellow signal alone is thereby warned that the related green signal is being terminated or that a red signal will be exhibited immediately thereafter and such vehicular traffic shall not enter or be crossing the intersection when the red signal is exhibited.
Maine (P)	If steady and circular or an arrow, means the operator must take warning that a green light is being terminated or a red light will be exhibited immediately.
Maryland (P)	Vehicular traffic facing a steady yellow signal is warned that the related green movement is ending or that a red signal, which will prohibit vehicular traffic from entering the intersection, will be shown immediately after the yellow signal.
Massachusetts (P)	*No specific information available, assume Uniform Vehicle Code as default.*
Michigan*	If the signal exhibits a steady yellow indication, vehicular traffic facing the signal shall stop before entering the nearest crosswalk at the intersection or at a limit line when marked, but if the stop cannot be made in safety, a vehicle may be driven cautiously through the intersection.
Minnesota (P)	Vehicular traffic facing a circular yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection, except for the continued movement allowed by any green arrow indication simultaneously exhibited.
Mississippi*	Vehicular traffic facing the signal shall stop before entering the nearest crosswalk at the intersection, but if such stop cannot be made in safety a vehicle may be driven cautiously through the intersection.
Missouri (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
Montana (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is warned that the traffic movement permitted by the related green signal is being terminated or that a red signal will be exhibited immediately thereafter. Vehicular traffic may not enter the intersection when the red signal is exhibited after the yellow signal.

States	Definition of yellow signal (for vehicles)
Nebraska*	Vehicular traffic facing a steady yellow indication is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection, and upon display of a steady yellow indication, vehicular traffic shall stop before entering the nearest crosswalk at the intersection, but if such stop cannot be made in safety, a vehicle may be driven cautiously through the intersection.
Nevada (P)	Vehicular traffic facing the signal is thereby warned that the related green movement is being terminated or that a steady red indication will be exhibited immediately thereafter, and such vehicular traffic must not enter the intersection when the red signal is exhibited.
New Hampshire (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection.
New Jersey*	Amber, or yellow, when shown alone following green means traffic to stop before entering the intersection or nearest crosswalk, unless when the amber appears the vehicle or street car is so close to the intersection that with suitable brakes it cannot be stopped in safety. A distance of 50 feet from the intersection is considered a safe stopping distance for a speed of 20 miles per hour, and vehicles and street cars if within that distance when the amber appears alone, and which cannot be stopped with safety, may proceed across the intersection or make a right or left turn unless the turning movement is specifically limited.
New Mexico (P)	Vehicular traffic facing the signal is warned that the red signal will be exhibited immediately thereafter and the vehicular traffic shall not enter the intersection when the red signal is exhibited except to turn as hereinafter provided.
New York (P)	Traffic, except pedestrians, facing a steady circular yellow signal may enter the intersection; however, said traffic is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
North Carolina (P)	When a traffic signal is emitting a steady yellow circular light on a traffic signal controlling traffic approaching an intersection or a steady yellow arrow light on a traffic signal controlling traffic turning at an intersection, vehicles facing the yellow light are warned that the related green light is being terminated or a red light will be immediately forthcoming.
North Dakota (P)	Vehicular traffic facing a steady circular yellow or yellow arrow indication is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic may not enter the intersection.
Ohio (P)	Vehicular traffic, streetcars, and trackless trolleys facing a steady circular yellow or yellow arrow signal are thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic, streetcars, and trackless trolleys shall not enter the intersection.
Oklahoma (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
Oregon*	A driver facing a steady circular yellow signal light is thereby warned that the related right-of-way is being terminated and that a red or flashing red light will be shown immediately. A driver facing the light shall stop at a clearly marked stop line, but if none, shall stop before entering the marked crosswalk on the near side of the intersection, or if there is no marked crosswalk, then before entering the intersection. If a driver cannot stop in safety, the driver may drive cautiously through the intersection.
Pennsylvania (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green indication is being terminated or that a red indication will be exhibited immediately thereafter.

States	Definition of yellow signal (for vehicles)
Rhode Island	Vehicular traffic facing the signal is warned by it that the red or "stop" signal will be exhibited immediately afterwards, and the vehicular traffic shall not enter or be crossing the intersection when the red or "stop" signal is exhibited.
South Carolina (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.
South Dakota (P)	Vehicular traffic facing the signal is thereby warned that the red or "stop" signal will be exhibited immediately thereafter and such vehicular traffic shall not enter the intersection when the red or "stop" signal is exhibited.
Tennessee	Vehicular traffic facing the signal is warned that the red or "Stop" signal will be exhibited immediately thereafter and that vehicular traffic shall not enter or cross the intersection when the red or "Stop" signal is exhibited.
Texas (P)	An operator of a vehicle facing a steady yellow signal is warned by that signal that: (1) movement authorized by a green signal is being terminated; or (2) a red signal is to be given.
Utah (P)	The operator of a vehicle facing a steady circular yellow or yellow arrow signal is warned that the allowable movement related to a green signal is being terminated.
Vermont (P)	Vehicular traffic facing a steady yellow signal is thereby warned that the related green signal is being terminated or that a red signal will be exhibited immediately thereafter, when vehicular traffic shall not enter the intersection.
Virginia*	Steady amber indicates that a change is about to be made in the direction of the moving of traffic. When the amber signal is shown, traffic which has not already entered the intersection, including the crosswalks, shall stop if it is not reasonably safe to continue, but traffic which has already entered the intersection shall continue to move until the intersection has been cleared. The amber signal is a warning that the steady red signal is imminent.
Washington (P)	Vehicle operators facing a steady circular yellow or yellow arrow signal are thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter when vehicular traffic shall not enter the intersection. Vehicle operators shall stop for pedestrians who are lawfully within the intersection control area as required by *RCW 46.61.235(1).*
West Virginia	Vehicular traffic facing the signal is thereby warned that the red or "stop" signal will be exhibited immediately thereafter and such vehicular traffic shall not enter or be crossing the intersection when the red or "stop" signal is exhibited.
Wisconsin*	When shown with or following the green, traffic facing a yellow signal shall stop before entering the intersection unless so close to it that a stop may not be made in safety.
Wyoming (P)	Vehicular traffic facing a steady circular yellow or yellow arrow signal is thereby warned that the related green movement is being terminated or that a red indication will be exhibited immediately thereafter.

(P) = “permissive” yellow law

*****States allowing intersection entry and clearance in situations where it is impossible or unsafe to stop are generally not in conflict with the “permissive” yellow law.

No indication = “restrictive” yellow law.

APPENDIX D

Traffic Signal Change Intervals Survey

The Institute of Transportation Engineers (ITE) is in the process of preparing Guidelines for Determining Traffic Signal Change Intervals, a Recommended Practice (RP). In 1985 ITE published a proposed recommended practice entitled *Determining Vehicle Change Intervals* that was not ratified to become an RP. Later, in 2001, ITE published the informational report *A History of the Yellow and All-Red Intervals for Traffic Signals*. In the interim, changes in technology, automated enforcement, the availability of new primary data, further research and the public and professional concern that a defined standard of reference does not exist with regard to this topic have led to the initiative to develop this RP.

This survey of transportation agencies is part of the effort to determine the current state-of-the-practice and to provide the user with an overview of key considerations to determine yellow change and red clearance intervals for traffic signals and their application.

SURVEY QUESTIONS

Please identify the location of your agency

Country State

- USA
- Canada
- Other (please specify):______

1. Does your agency have a formal policy for timing the traffic signal changes intervals?

- Yes
- No

Is there a formal policy for the use of the optional all-red interval?

- Yes
- No

If yes to either question, please submit material via email to dnoble@ite.org. *(Note: An email address will need to be provided.)*

Name _______

Agency _______

Telephone ______

Email _________

2. If there is no formal policy, generally what method do you use to determine the duration of change intervals?

a. The following kinematic equation is used:

$$CP = t + \frac{V}{2a + 64.4g} + \frac{W + L}{V}$$

b. A uniform value is used for all intersections (e.g., 4 seconds).

c. A uniform value is used for all intersections (e.g., 4 seconds), except where conditions warrant an exception to the uniform timing.

d. A table of values by approach speed is applied to all intersections.

e. Other (please specify):______

3. What, if any, are your minimum and maximum values for the yellow intervals, all-red intervals, and total change interval?

Yellow min:	_____	All-red min:	_____	Total interval min:	____
Yellow max:	_____	All-red max:	______	Total interval max:	___

4. If you use the kinematic equation displayed in question 2, how do you allocate time between the yellow and all-red interval?

a. The calculated value from the first two terms of the equation is allocated to the yellow interval and the third term is allocated to the all-red interval.

b. The yellow interval is set at a uniform duration (e.g., 4 seconds) and the remainder is allocated to the all-red interval.

c. The all-red interval is set at a uniform duration (e.g., 1 second) and the remainder is allocated to the yellow interval.

d. The entire time is allocated to the yellow interval. The all-red interval is not used.

e. Other (please specify):______

5. If you use an equation similar to the kinematic equation in question 2, what values do you use for the following variables?

PRT (t)=

Deceleration (a) =

Vehicle length (l) =

6a. If speed is used to calculate the interval durations, what speed do you use?

a. 85th percentile approach speed:

b. Posted speed limit

c. Design speed

d. Other (specify):_____

6b. If a different speed is used to calculate the all-red interval, what speed do you use (for example, some agencies used 85th percentile speed to time the yellow interval and posted speed to time the all-red interval)?

a. 85th percentile approach speed:

b. Posted speed limit

c. Design speed

d. Other (specify):_____

7. If speed measurements are collected in the field, how frequently are they updated?

a. Not collected

b. Only once to time the interval

c. Annually

d. As conditions change

e. Other (specify):_____

8. Other than speed, do you collect any field measurements (e.g., intersection width, pedestrian volumes) prior to timing the change interval?

9. Do you have a procedure for special situations (e.g. left- or right-turn signals) or for special populations (e.g. large trucks, bicyclists, transit vehicles with standing passengers)?

10. Comments or additional information.

APPENDIX E

Detailed Site Characteristics

Michigan Study-Site Characteristics

Subject Approach	Cross Street	Dir.	City	Speed Limit (mph)	Yellow Duration (s)	All-Red Duration (s)	Cycle Length (s)	Clearing Width (ft)	Area Type	Grade	Red-Light Cameras
Anthony Wayne	Kirby	NB	Detroit	25	4.0	1.0	<90	≤ 48	Urban	Level	None
Anthony Wayne	Kirby	SB	Detroit	25	4.0	1.0	<90	≤ 48	Urban	Level	None
Warren	Cass	EB	Detroit	30	4.0	1.0	<90	48-72	Urban	Level	None
Warren	Anthony Wayne	WB	Detroit	30	4.5	1.5	<90	>120	Urban	Level	None
Woodward	Kirby	NB	Detroit	30	3.6	2.5	90-120	96-120	Urban	Level	None
6-Mile	Newburgh	WB	Livonia	45	4.5	0.0	90-120	96-120	Suburban	Level	None
Haggerty	College Pkwy	NB	Livonia	45	4.3	1.1	<90	≤ 48	Suburban	Level	None
Haggerty	6-Mile	SB	Livonia	45	4.7	0.0	<90	72-96	Suburban	Level	None
Newburgh	6-Mile	NB	Livonia	45	5.0	0.0	90-120	48-72	Suburban	Level	None
6-Mile	Haggerty	EB	Northville Twp	45	4.8	0.0	<90	96-120	Suburban	Level	None
7-Mile	Haggerty	EB	Northville Twp	45	5.0	0.0	<90	96-120	Suburban	Level	None
Beck (south)	Compuware	NB	Plymouth Twp	45	4.3	1.5	<90	72-96	Suburban	Level	None
Beck (south)	Territorial	SB	Plymouth Twp	45	4.3	1.4	90-120	72-96	Suburban	Level	None
Plymouth	Earhart	WB	Ann Arbor	50	3.5	2.5	<90	96-120	Rural	Level	None
Ellsworth	Carpenter	WB	Pittsfield Twp	45	4.3	2.3	<90	96-120	Suburban	Level	None
Packard	Carpenter	EB	Pittsfield Twp	45	4.3	2.0	120-180	72-96	Suburban	Level	None
Packard	Carpenter	WB	Pittsfield Twp	45	4.2	2.0	120-180	72-96	Suburban	Level	None
Carpenter	Center Valley	NB	Pittsfield Twp	45	4.3	1.4	<90	72-96	Suburban	Level	None
Carpenter	Center Valley	SB	Pittsfield Twp	45	4.3	1.4	<90	48-72	Suburban	Level	None
Ford	Plymouth	SB	Superior Twp	55	5.1	1.5	<90	72-96	Rural	Level	None
Beck (north)	Grand River	SB	Novi	40	3.9	2.0	120-180	>120	Suburban	Level	None
Novi	Grand River	NB	Novi	40	3.9	2.0	120-180	>120	Suburban	Level	None
Grand River	Novi	EB	Novi	40	3.9	2.0	120-180	96-120	Suburban	Level	None

Note: Dir. = direction.

Florida Study-Site Characteristics

Subject Approach	Cross Street	Dir.	City	Speed Limit (mph)	Yellow Duration (s)	All-Red Duration (s)	Cycle Length (s)	Clearing Width (ft)	Area Type	Grade	Red-Light Cameras
Central Florida Pkwy	Westwood	EB	Orlando	45	4.3	1.0	<90	>120	Suburban	Level	None
Central Florida Pkwy	International	WB	Orlando	45	4.3	1.0	120-180	>120	Suburban	Level	None
International	Central Florida Pkwy	NB	Orlando	45	4.3	1.0	120-180	>120	Suburban	Level	None
International	Sea Harbor	SB	Orlando	45	4.3	1.0	120-180	>120	Suburban	Level	None
John Young	Commerce Park	SB	Orlando	55	5.0	2.0	120-180	96-120	Suburban	Level	None
Kirkman	Conroy	NB	Orlando	45	5.0	2.0	>180	>120	Suburban	Level	None
Kirkman	Conroy	SB	Orlando	45	5.0	2.0	>180	>120	Suburban	Level	None
Kirkman	Vineland	NB	Orlando	50	4.8	2.2	120-180	96-120	Suburban	Level	None
International	Sand Lake	NB	Orlando	40	4.0	1.0	120-180	>120	Suburban	Level	None
Sand Lake	International	WB	Orlando	40	4.0	1.0	120-180	72-96	Suburban	Level	None
Sand Lake	Orange Blossom	EB	Orlando	45	4.3	1.0	120-180	>120	Suburban	Level	None
Orange Blossom	Sand Lake	SB	Orlando	45	4.3	1.0	120-180	>120	Suburban	Level	None
SR 535	Hotel Plaza	NB	Lake Buena Vista	40	4.0	1.0	120-180	>120	Suburban	Level	None
SR 535	Hotel Plaza	SB	Lake Buena Vista	40	4.0	1.0	120-180	>120	Suburban	Level	None
SR 535	I-4 EB	SB	Lake Buena Vista	40	4.3	1.0	120-180	>120	Suburban	Level	None
US 441	NE 136th Ave	NB	Lady Lake (The Villages)	45	5.0	2.0	120-180	72-96	Rural	Level	None
US 441	NE 136th Ave	SB	Lady Lake (The Villages)	45	5.0	2.0	120-180	72-96	Rural	Level	None
US 441	Avenida Central/Griffin	NB	Lady Lake (The Villages)	45	6.0	1.0	120-180	48-72	Rural	Level	None
CR 466	Belvedere/CR101	WB	Lady Lake (The Villages)	45	4.0	1.5	120-180	96-120	Rural	Level	None
CR 466	Southern Trace	EB	Lady Lake (The Villages)	45	4.0	1.5	90-120	96-120	Rural	Level	None

California Study-Site Characteristics

Subject Approach	Cross Street	Dir.	City	Speed Limit (mph)	Yellow Duration (s)	All-Red Duration (s)	Cycle Length (s)	Clearing Width (ft)	Area Type	Grade	Red-Light Cameras
State College	Lincoln	NB	Anaheim	40	3.5	1.0	90-120	>120	Suburban	Level	None
Lincoln	State College	WB	Anaheim	40	3.5	1.0	90-120	>120	Suburban	Level	None
Ball	East	WB	Anaheim	40	4.5	1.0	<90	72-96	Suburban	Level	None
Ball	East	EB	Anaheim	40	4.5	1.0	<90	96-120	Suburban	Level	None
Imperial (central)	Santa Ana Canyon	SB	Anaheim	40	4.0	1.0	120-180	>120	Suburban	4.20%	None
Imperial (north)	LaPalma	NB	Anaheim	40	4.5	1.0	120-180	96-120	Suburban	Level	None
Katela	State College	WB	Anaheim	40	4.0	1.0	120-180	>120	Suburban	Level	None
Imperial (central)	Canyon H.S.	SB	Anaheim	40	4.0	1.0	120-180	48-72	Suburban	Level	None
Imperial (central)	Santa Ana Canyon	NB	Anaheim	40	4.0	1.0	90-120	>120	Suburban	Level	None
Imperial (south)	Nohl Ranch	NB	Anaheim	40	4.5	1.0	<90	72-96	Suburban	-7.30%	None
Nohl Ranch	Imperial	WB	Anaheim	40	4.0	1.0	<90	>120	Suburban	Level	None
Jamboree (west)	Barranca	WB	Irvine	60	5.0	2.0	120-180	>120	Suburban	Level	None
Jamboree (west)	Barranca	EB	Irvine	50	5.0	2.0	120-180	96-120	Suburban	Level	None
Jamboree (east)	Walnut	EB	Irvine	60	4.0	2.0	<90	>120	Suburban	Level	None
Westminster	Harbor	WB	Santa Ana	45	4.5	1.0	120-180	96-120	Suburban	Level	Present
Dyer	Pullman	WB	Santa Ana	40	4.0	1.0	120-180	48-72	Suburban	Level	Present
Whittier	Atlantic	WB	E. Los Angeles	30	4.0	0.0	90-120	72-96	Urban	Level	Present
Colima	Batson	WB	Rowland Hts	40	4.5	0.0	90-120	48-72	Suburban	Level	Present
Fullerton	Pathfinder	NB	Rowland Hts	50	5.0	0.0	<90	72-96	Suburban	-4.70%	None
Fullerton	Pathfinder	SB	Rowland Hts	45	5.0	0.0	<90	72-96	Suburban	5.70%	None
Telegraph	Colima	SB	Whittier	45	5.0	1.0	90-120	72-96	Suburban	Level	Present

Virginia Study-Site Characteristics

Subject Approach	Cross Street	Dir.	City	Speed Limit (mph)	Yellow Duration (s)	All-Red Duration (s)	Cycle Length (s)	Clearing Width (ft)	Area Type	Grade	Red-Light Cameras
Lee Jackson Mem. Hwy	Loudoun Co. Pkwy	EB	Sterling	55	5.5	3.0	120-180	>120	Suburban	Level	None
Lee Jackson Mem. Hwy	Loudoun Co. Pkwy	WB	Sterling	55	5.5	3.0	120-180	>120	Suburban	Level	None
Lee Jackson Mem. Hwy	Walney	WB	Chantilly	45	5.5	3.0	>180	>120	Suburban	Level	None
Leesburg Pike	Colvin Run	EB	Great Falls	55	5.5	4.5	>180	48-72	Suburban	Level	None
Leesburg Pike	Colvin Run	WB	Great Falls	55	5.5	4.5	>180	72-96	Suburban	Level	None
Leesburg Pike	Countryside	EB	Sterling	50	5.0	2.0	>180	96-120	Suburban	Level	None
Leesburg Pike	Countryside	WB	Sterling	50	5.0	2.0	>180	>120	Suburban	6.25%	None
Fairfax Co. Pkwy	West Ox	NB	Herndon	50	5.0	2.0	>180	>120	Suburban	Level	None
Fairfax Co. Pkwy	West Ox	SB	Herndon	50	5.0	2.0	>180	>120	Suburban	Level	None
Fairfax Co. Pkwy	Fox Mill	NB	Reston	50	5.0	2.0	>180	>120	Suburban	Level	None
Fairfax Co. Pkwy	Fox Mill	EB	Reston	35	4.0	3.0	>180	>120	Suburban	Level	None

Maryland Study-Site Characteristics

Subject Approach	Cross Street	Dir.	City	Speed Limit (mph)	Yellow Duration (s)	All-Red Duration (s)	Cycle Length (s)	Clearing Width (ft)	Area Type	Grade	Red-Light Cameras
Snowden River Pkwy	Oakland Mills	EB	Columbia	45	4.5	1.5	120-180	72-96	Suburban	Level	Present
Snowden River Pkwy	Oakland Mills	WB	Columbia	45	4.5	1.5	120-180	96-120	Suburban	Level	Present
Broken Land Pkwy	Cradlerock Way North	NB	Columbia	45	4.5	2.0	90-120	72-96	Suburban	Level	None
Broken Land Pkwy	Cradlerock Way North	SB	Columbia	45	4.5	2.0	90-120	72-96	Suburban	Level	Present
Georgia Ave	Norbeck	SB	Silver Spring	50	5.0	2.0	>180	>120	Suburban	Level	Present
Georgia Ave	Norbeck	EB	Silver Spring	40	4.0	2.0	>180	>120	Suburban	Level	None
Viers Mill Rd	Twinbrook	NB	Rockville	45	4.5	3.0	120-180	72-96	Suburban	Level	None
Viers Mill Rd	Twinbrook	SB	Rockville	40	4.5	3.0	120-180	72-96	Suburban	Level	Present

APPENDIX F

Effect of 1-Second Red Clearance Interval Reduction on Intersection Clearance

Posted Speed Limit + 7 mph (estimate of the 85th percentile speed)

The tables show the calculated red clearance interval (in seconds) for the approach speed estimation based on the respective posted speed limit (V) plus 7 mph and given intersection width (W). The intersection width has been increased to account for stop line setback. The first table represents the minimum stop line setback of 4 feet and the second table represents the maximum stop line setback of 30 feet. The calculated values assume a vehicle length (L) of 20 feet and include the 1-second reduction. The highlighted intervals have been increased to the minimum recommended value of 1.0 seconds.

Calculated red clearance interval considering 4-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit +7 mph, V (fps)	Width, W (ft)								
		28	40	52	64	76	88	100	112	124
25	47.04	1.0	1.0	1.0	1.0	1.0	1.3	1.6	1.8	2.1
30	54.39	1.0	1.0	1.0	1.0	1.0	1.0	1.2	1.4	1.6
35	61.74	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.3
40	69.09	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.1
45	76.44	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
50	83.79	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
55	91.14	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

Calculated red clearance interval considering 30-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit +7 mph, V (fps)	Width, W (ft)								
		54	66	78	90	102	114	126	138	150
25	47.04	1.0	1.0	1.1	1.3	1.6	1.8	2.1	2.4	2.6
30	54.39	1.0	1.0	1.0	1.0	1.2	1.5	1.7	1.9	2.1
35	61.74	1.0	1.0	1.0	1.0	1.0	1.2	1.4	1.6	1.8
40	69.09	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.3	1.5
45	76.44	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.2
50	83.79	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
55	91.14	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

The next tables show the front bumper position (in feet) of a vehicle passing through an intersection at the end of the red clearance interval. This was calculated by multiplying the red clearance interval from the previous tables (in seconds) by the posted speed limit (in feet per second). The first table represents the minimum stop line setback of 4 feet and the second table represents the maximum stop line setback of 30 feet.

Front bumper position at the end of the red clearance interval considering 4-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit +7 mph, V (fps)	Width, W (ft)								
		28	40	52	64	76	88	100	112	124
25	47.04	47.04	47.04	47.04	47.04	48.96	60.96	72.96	84.96	96.96
30	54.39	54.39	54.39	54.39	54.39	54.39	53.61	65.61	77.61	89.61
35	61.74	61.74	61.74	61.74	61.74	61.74	61.74	61.74	70.26	82.26
40	69.09	69.09	69.09	69.09	69.09	69.09	69.09	69.09	69.09	74.91
45	76.44	76.44	76.44	76.44	76.44	76.44	76.44	76.44	76.44	76.44
50	83.79	83.79	83.79	83.79	83.79	83.79	83.79	83.79	83.79	83.79
55	91.14	91.14	91.14	91.14	91.14	91.14	91.14	91.14	91.14	91.14

Front bumper position at the end of the red clearance interval considering 30-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit +7 mph, V (fps)	Width, W (ft)								
		54	66	78	90	102	114	126	138	150
25	47.04	47.04	47.04	50.96	62.96	74.96	86.96	98.96	110.96	122.96
30	54.39	54.39	54.39	54.39	55.61	67.61	79.61	91.61	103.61	115.61
35	61.74	61.74	61.74	61.74	61.74	60.26	72.26	84.26	96.26	108.26
40	69.09	69.09	69.09	69.09	69.09	69.09	69.09	76.91	88.91	100.91
45	76.44	76.44	76.44	76.44	76.44	76.44	76.44	76.44	81.56	93.56
50	83.79	83.79	83.79	83.79	83.79	83.79	83.79	83.79	83.79	86.21
55	91.14	91.14	91.14	91.14	91.14	91.14	91.14	91.14	91.14	91.14

The final tables show the difference between the front bumper position at the end of the red clearance interval and the intersection width (in feet). The highlighted values in the lower left represent the distances that place the clearing driver entirely beyond the intersection during the red clearance interval. The values that are not highlighted represent the distances that place the clearing driver entirely beyond the intersection at some instance during the 1 second of start-up delay. The highlighted values in the upper right represent the distances that place the rear bumper of the clearing driver exactly at the intersection width at the end of the 1 second of start-up delay. The first table represents the minimum stop line setback of 4 feet and the second table represents the maximum stop line setback of 30 feet.

Difference between front bumper position and intersection width considering 4-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit +7 mph, V (fps)	Width, W (ft)								
		28	40	52	64	76	88	100	112	124
25	47.04	19.04	7.04	-4.96	-16.96	-27.04	-27.04	-27.04	-27.04	-27.04
30	54.39	26.39	14.39	2.39	-9.61	-21.61	-34.39	-34.39	-34.39	-34.39
35	61.74	33.74	21.74	9.74	-2.26	-14.26	-26.26	-38.26	-41.74	-41.74
40	69.09	41.09	29.09	17.09	5.09	-6.91	-18.91	-30.91	-42.91	-49.09
45	76.44	48.44	36.44	24.44	12.44	0.44	-11.56	-23.56	-35.56	-47.56
50	83.79	55.79	43.79	31.79	19.79	7.79	-4.21	-16.21	-28.21	-40.21
55	91.14	63.14	51.14	39.14	27.14	15.14	3.14	-8.86	-20.86	-32.86

Difference between front bumper position and intersection width considering 30-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit +7 mph, V (fps)	Width, W (ft)								
		54	66	78	90	102	114	126	138	150
25	47.04	-6.96	-18.96	-27.04	-27.04	-27.04	-27.04	-27.04	-27.04	-27.04
30	54.39	0.39	-11.61	-23.61	-34.39	-34.39	-34.39	-34.39	-34.39	-34.39
35	61.74	7.74	-4.26	-16.26	-28.26	-41.74	-41.74	-41.74	-41.74	-41.74
40	69.09	15.09	3.09	-8.91	-20.91	-32.91	-44.91	-49.09	-49.09	-49.09
45	76.44	22.44	10.44	-1.56	-13.56	-25.56	-37.56	-49.56	-56.44	-56.44
50	83.79	29.79	17.79	5.79	-6.21	-18.21	-30.21	-42.21	-54.21	-63.79
55	91.14	37.14	25.14	13.14	1.14	-10.86	-22.86	-34.86	-46.86	-58.86

In all scenarios, the clearing driver has exited the intersection by the end of the 1-second of start-up delay. The analysis shows that more drivers are able to traverse beyond the intersection during the red clearance interval and fewer drivers are clearing the intersection at the end of the 1-second start-up delay when stop line setback is the minimum distance of 4 feet. The trend reverses as the stop line setback increases. Example cases for a 4-foot stop line setback are shown graphically in the following figures, representing each of the “highlighted” areas discussed in the final tables.

Example 1: Driver is beyond intersection during red clearance interval.

W = 24-ft intersection width + 4-ft stop line setback = 28 ft

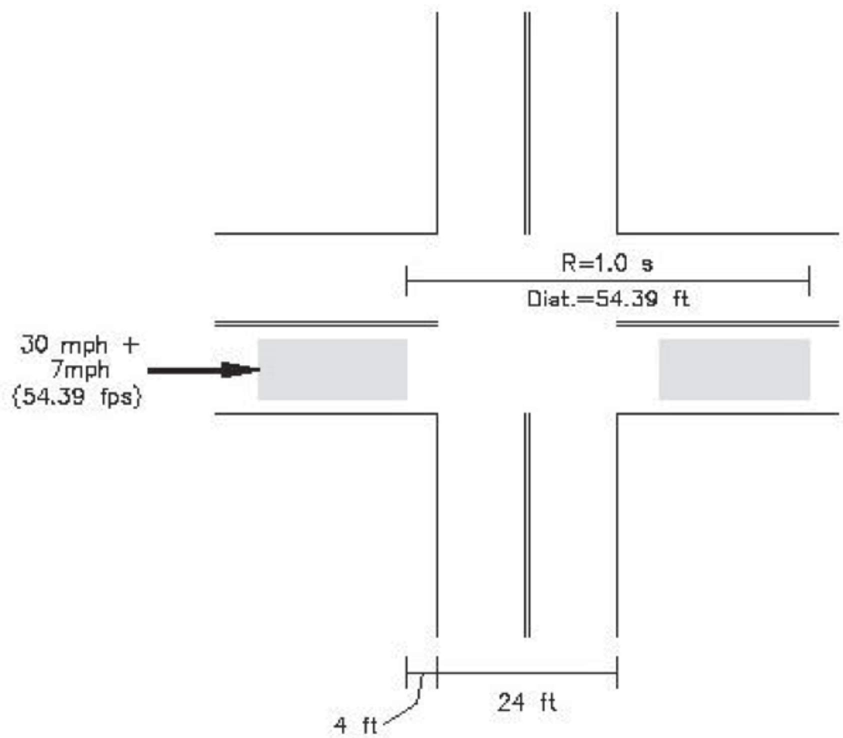
L = 20 ft

V = 30 mph + 7 mph = 37 mph (54.39 fps)

$$\frac{W+L}{V} - 1 = \frac{28\,ft + 20\,ft}{54.39\,fps} - 1 = -0.12\,s \qquad \text{(1.0-second minimum)}$$

$$(1.0\,s) * \; 54.39\,fps \; = \; 54.39\,ft$$

$$54.39\,ft - 28\,ft = 26.39\,ft$$



Example 2: Driver is beyond intersection during start-up delay.

W = 72-ft intersection width + 4-ft stop line setback = 76 ft

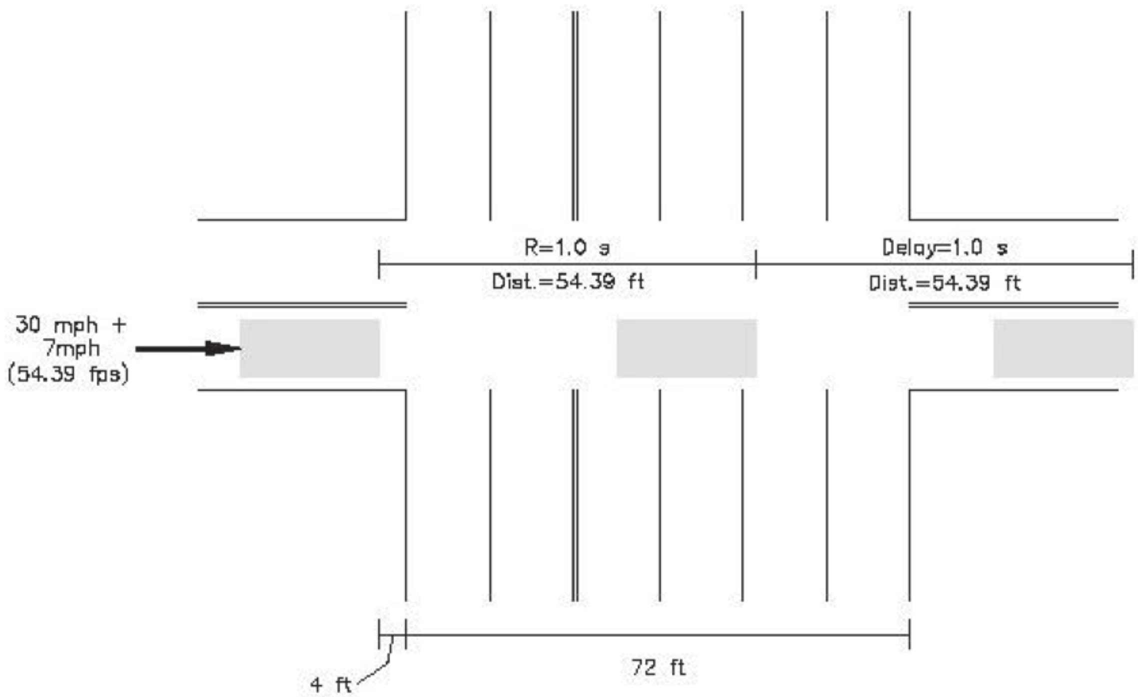
L = 20 ft

V = 30 mph + 7 mph = 37 mph (54.39 fps)

$$\frac{W+L}{V} - 1 = \frac{76\,ft + 20\,ft}{54.39\,fps} - 1 = 0.77\,s \qquad \text{(1.0-second minimum)}$$

$$(1.0\,s) * 54.39\,fps = 54.39\,ft$$

$$54.39\,ft - 76\,ft = -21.61\,ft$$



Example 3: Driver clears intersection at end of start-up delay.

W = 108-ft intersection width + 4-ft stop line setback = 112 ft

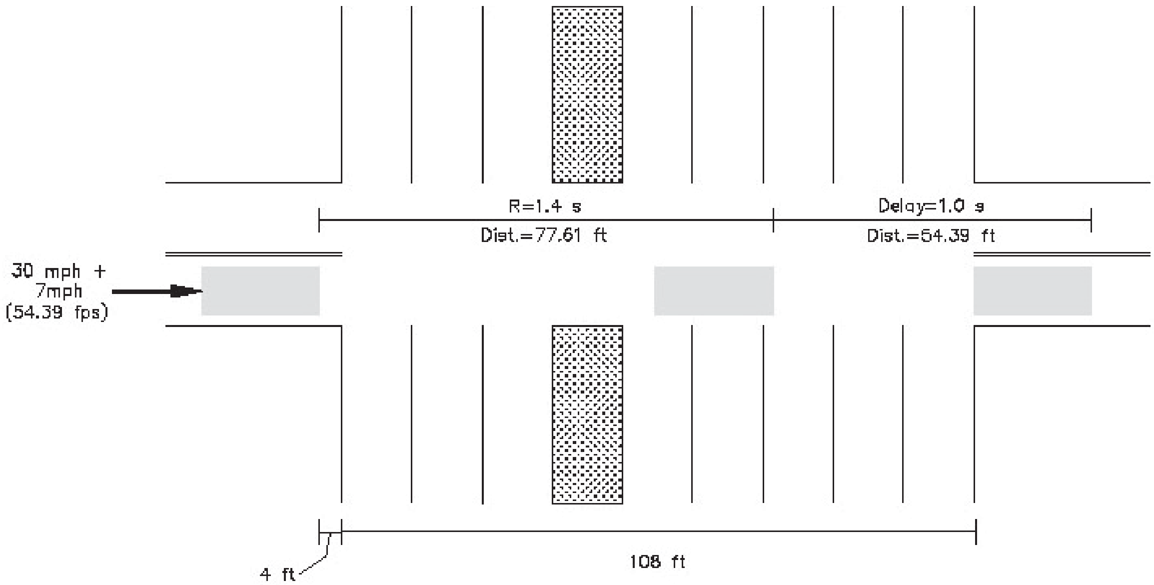
L = 20 ft

V = 30 mph + 7 mph = 37 mph (54.39 fps)

$$\frac{W+L}{V} - 1 = \frac{112\,ft + 20\,ft}{54.39\,fps} - 1 = 1.43\,s$$

$$1.4\,s * 54.39\,fps = 77.61\,ft$$

$$77.61\,ft - 112\,ft = -34.39\,ft$$



Posted Speed Limit

The tables show the calculated red clearance interval (in seconds) assuming the approach speed is assumed to equal the respective posted speed limit (V) and given intersection width (W). The intersection width has been increased to account for stop line setback. The first table represents the minimum stop line setback of 4 feet and the second table represents the maximum stop line setback of 30 feet. The calculated values assume a vehicle length (L) of 20 feet and include the 1-second reduction. The highlighted intervals have been increased to the minimum recommended value of 1.0 seconds.

Calculated red clearance interval considering 4-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit, V (fps)	Width, W (ft)								
		28	40	52	64	76	88	100	112	124
25	36.75	1.0	1.0	1.0	1.3	1.6	1.9	2.3	2.6	2.9
30	44.1	1.0	1.0	1.0	1.0	1.2	1.4	1.7	2.0	2.3
35	51.45	1.0	1.0	1.0	1.0	1.0	1.1	1.3	1.6	1.8
40	58.8	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.2	1.4
45	66.15	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.2
50	73.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
55	80.85	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

Calculated red clearance interval considering 30-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit, V (fps)	Width, W (ft)								
		54	66	78	90	102	114	126	138	150
25	36.75	1.0	1.3	1.7	2.0	2.3	2.6	3.0	3.3	3.6
30	44.1	1.0	1.0	1.2	1.5	1.8	2.0	2.3	2.6	2.9
35	51.45	1.0	1.0	1.0	1.1	1.4	1.6	1.8	2.1	2.3
40	58.8	1.0	1.0	1.0	1.0	1.1	1.3	1.5	1.7	1.9
45	66.15	1.0	1.0	1.0	1.0	1.0	1.0	1.2	1.4	1.6
50	73.5	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.3
55	80.85	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.1

The next tables show the front bumper position (in feet) of a vehicle passing through an intersection during the red clearance interval. This was calculated by multiplying the red clearance interval from the previous tables (in seconds) by the posted speed limit (in feet per second). The first table represents the minimum stop line setback of 4 feet and the second table represents the maximum stop line setback of 30 feet.

Front bumper position at the end of the red clearance interval considering 4-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit, V (fps)	Width, W (ft)								
		28	40	52	64	76	88	100	112	124
25	36.75	36.75	36.75	35.25	47.25	59.25	71.25	83.25	95.25	107.25
30	44.1	44.10	44.10	44.10	44.10	51.90	63.90	75.90	87.90	99.90
35	51.45	51.45	51.45	51.45	51.45	51.45	56.55	68.55	80.55	92.55
40	58.8	58.80	58.80	58.80	58.80	58.80	58.80	61.20	73.20	85.20
45	66.15	66.15	66.15	66.15	66.15	66.15	66.15	66.15	65.85	77.85
50	73.5	73.50	73.50	73.50	73.50	73.50	73.50	73.50	73.50	70.50
55	80.85	80.85	80.85	80.85	80.85	80.85	80.85	80.85	80.85	80.85

Front bumper position at the end of the red clearance interval considering 30-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit, V (fps)	Width, W (ft)								
		54	66	78	90	102	114	126	138	150
25	36.75	37.25	49.25	61.25	73.25	85.25	97.25	109.25	121.25	133.25
30	44.1	44.10	41.90	53.90	65.90	77.90	89.90	101.90	113.90	125.90
35	51.45	51.45	51.45	51.45	58.55	70.55	82.55	94.55	106.55	118.55
40	58.8	58.80	58.80	58.80	58.80	63.20	75.20	87.20	99.20	111.20
45	66.15	66.15	66.15	66.15	66.15	66.15	67.85	79.85	91.85	103.85
50	73.5	73.50	73.50	73.50	73.50	73.50	73.50	72.50	84.50	96.50
55	80.85	80.85	80.85	80.85	80.85	80.85	80.85	80.85	77.15	89.15

The final tables show the difference between the front bumper position at the end of the red clearance interval and the intersection width remaining (in feet). The highlighted values in the lower left represent the distances that place the clearing driver entirely beyond the intersection during the red clearance interval. The values that are not highlighted represent the distances that place the clearing driver entirely beyond the intersection at some instance during the 1 second of start-up delay. The highlighted values in the upper right represent the distances that place the rear bumper of the clearing driver exactly at the intersection width at the end of the 1 second of start-up delay. The first table represents the minimum stop line setback of 4 feet and the second table represents the maximum stop line setback of 30 feet.

Difference between front bumper position and intersection width considering 4-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit, V (fps)	Width, W (ft)								
		28	40	52	64	76	88	100	112	124
25	36.75	8.75	-3.25	-16.75	-16.75	-16.75	-16.75	-16.75	-16.75	-16.75
30	44.1	16.10	4.10	-7.90	-19.90	-24.10	-24.10	-24.10	-24.10	-24.10
35	51.45	23.45	11.45	-0.55	-12.55	-24.55	-31.45	-31.45	-31.45	-31.45
40	58.8	30.80	18.80	6.80	-5.20	-17.20	-29.20	-38.80	-38.80	-38.80
45	66.15	38.15	26.15	14.15	2.15	-9.85	-21.85	-33.85	-46.15	-46.15
50	73.5	45.50	33.50	21.50	9.50	-2.50	-14.50	-26.50	-38.50	-53.50
55	80.85	52.85	40.85	28.85	16.85	4.85	-7.15	-19.15	-31.15	-43.15

Difference between front bumper position and intersection width considering 30-foot stop line setback

Posted Speed Limit, V (mph)	Posted Speed Limit, V (fps)	Width, W (ft)								
		54	66	78	90	102	114	126	138	150
25	36.75	-16.75	-16.75	-16.75	-16.75	-16.75	-16.75	-16.75	-16.75	-16.75
30	44.1	-9.90	-24.10	-24.10	-24.10	-24.10	-24.10	-24.10	-24.10	-24.10
35	51.45	-2.55	-14.55	-26.55	-31.45	-31.45	-31.45	-31.45	-31.45	-31.45
40	58.8	4.80	-7.20	-19.20	-31.20	-38.80	-38.80	-38.80	-38.80	-38.80
45	66.15	12.15	0.15	-11.85	-23.85	-35.85	-46.15	-46.15	-46.15	-46.15
50	73.5	19.50	7.50	-4.50	-16.50	-28.50	-40.50	-53.50	-53.50	-53.50
55	80.85	26.85	14.85	2.85	-9.15	-21.15	-33.15	-45.15	-60.85	-60.85

Similar to the estimated 85th percentile speed analysis, the clearing driver has exited the intersection by the end of the 1-second of start-up delay in all scenarios. Additionally, the analysis shows that more drivers are able to traverse beyond the intersection during the red clearance interval and fewer drivers are clearing the intersection at the end of the 1-second start-up delay when stop line setback is the minimum distance of 4 feet. The trend reverses as the stop line setback increases. Example cases are not included for this analysis, as the theory does not change.

Abbreviations and acronyms used without definitions in TRB publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation